Parabolic Slab

Arch Slab

نسألكم الدعاء

*IF you download the Free **APP.*** RC Structures YASSER EL LEATHY *on your smart phone or tablet, you will be able to play illustrative movies For any paragraph that has a QR code icon*

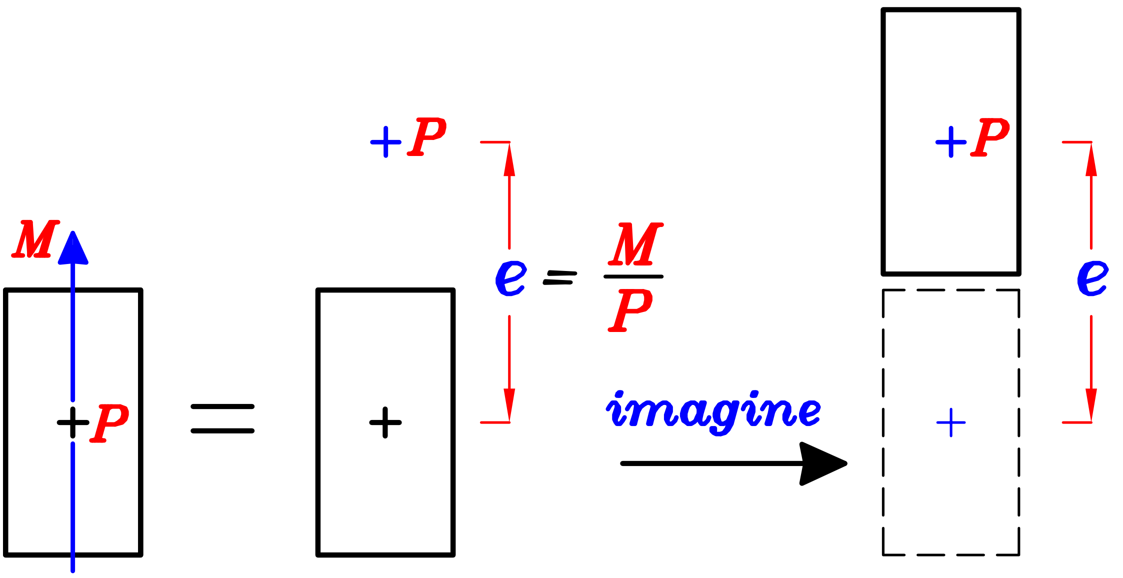
اذا حملت تطبيق RC Structures YASSER EL LEATHY على تليفونك المحمول او اللوح السطحى ستستطيع أن تشغل أفلام شرح للمقاطع التى تحتوى على رمز

Parabolic Slab. Table of Contents.

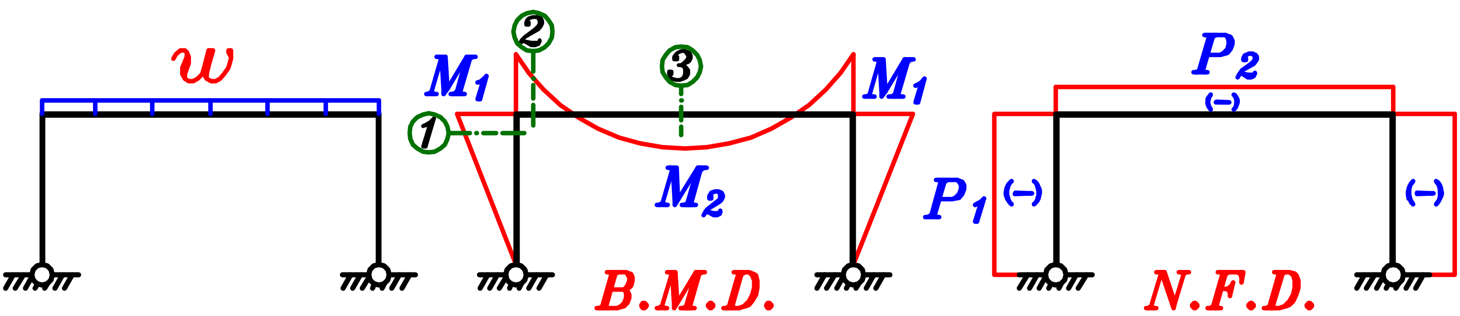
Introduction.

Thrust Line. (Pressure Line).

للقطاعات المؤثر عليها M , P اذا تخيلنا أنه تم ترحيل القطاع مسافه e عكس اتجاه الـ moment سيكون القطاع المرحل عليه Normal Force فقط وبالتالى عند تصميمه سيحتاج ابعاد قطاع اقل و كميه حديد تسليح اقل .



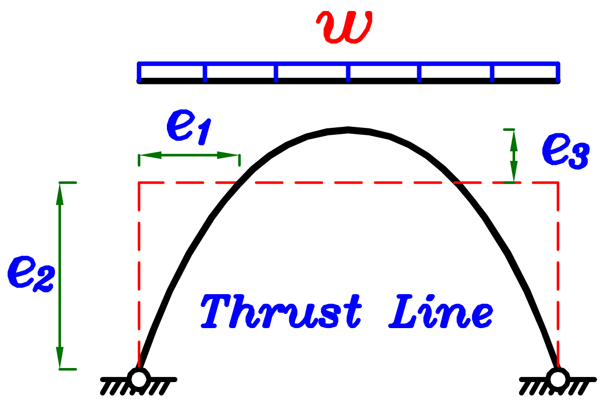
اذا استطعنا لاى structure ان نرحل كل قطاعاته عكس اتجاه الـ moment مسافه e سنضمن ان الـ structure الجديد كل قطاعاته سيؤثر عليها Normal Force فقط.
و بالتالى تكون ابعاد قطاعاته و كميات حديد تسليحه اقل فتكون تكلفته أقل .
و يسمى الـ structure الجديد Thrust Line أو Pressure Line .



Sec. ① $e_1 = \frac{M_1}{P_1}$

Sec. ② $e_2 = \frac{M_1}{P_2}$

Sec. ③ $e_3 = \frac{M_2}{P_2}$



المنشأت التى شكلها نفس شكل (*Thrust Line*)

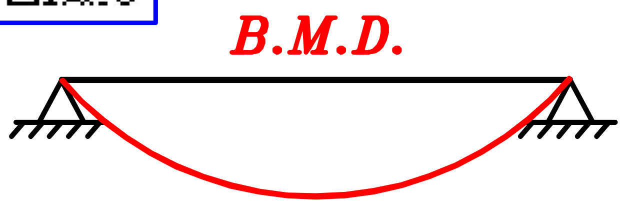
و لان فى هذه المنشأت تكون قيمه (*axial Force*) **تقريبا** ثابته على جميع القطاعات .

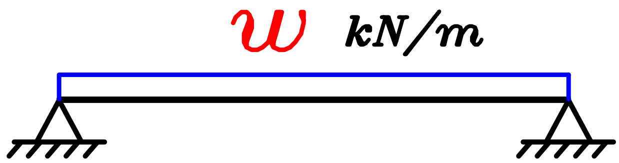
أى أن $\left(e = \frac{M}{P} = \frac{M}{constant} \right)$

لذا اذا رسمنا شكل الـ (*structure*) عكس شكل الـ (*B.M.D.*) يكون هو نفسه
شكل الـ (*Thrust Line*) أى لا يكون عليه (*Bending moment*)
و لكن يؤثر عليه فقط (*axial Force*) .

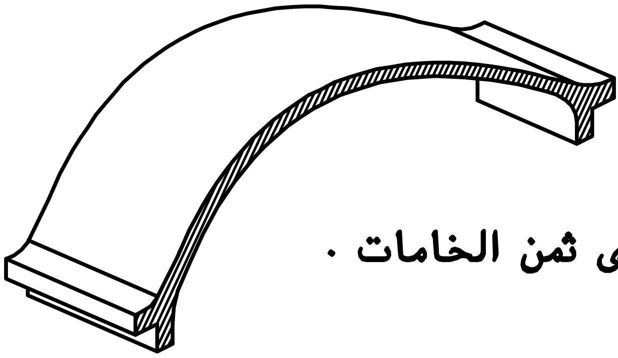
و هذه تعتبر ميزه اقتصاديه لان هذا يوفر فى كميات كلا من الخرسانه و حديد التسليح .

لان البلاطه تحمل احمال منتظمه فيكون شكل الـ (*Bending moment*) عباره عن *parabola*





فيفضل اخذ البلاطه *parabola* و لكن لاعلى لكى يكون عكس الـ *B.M.D.*

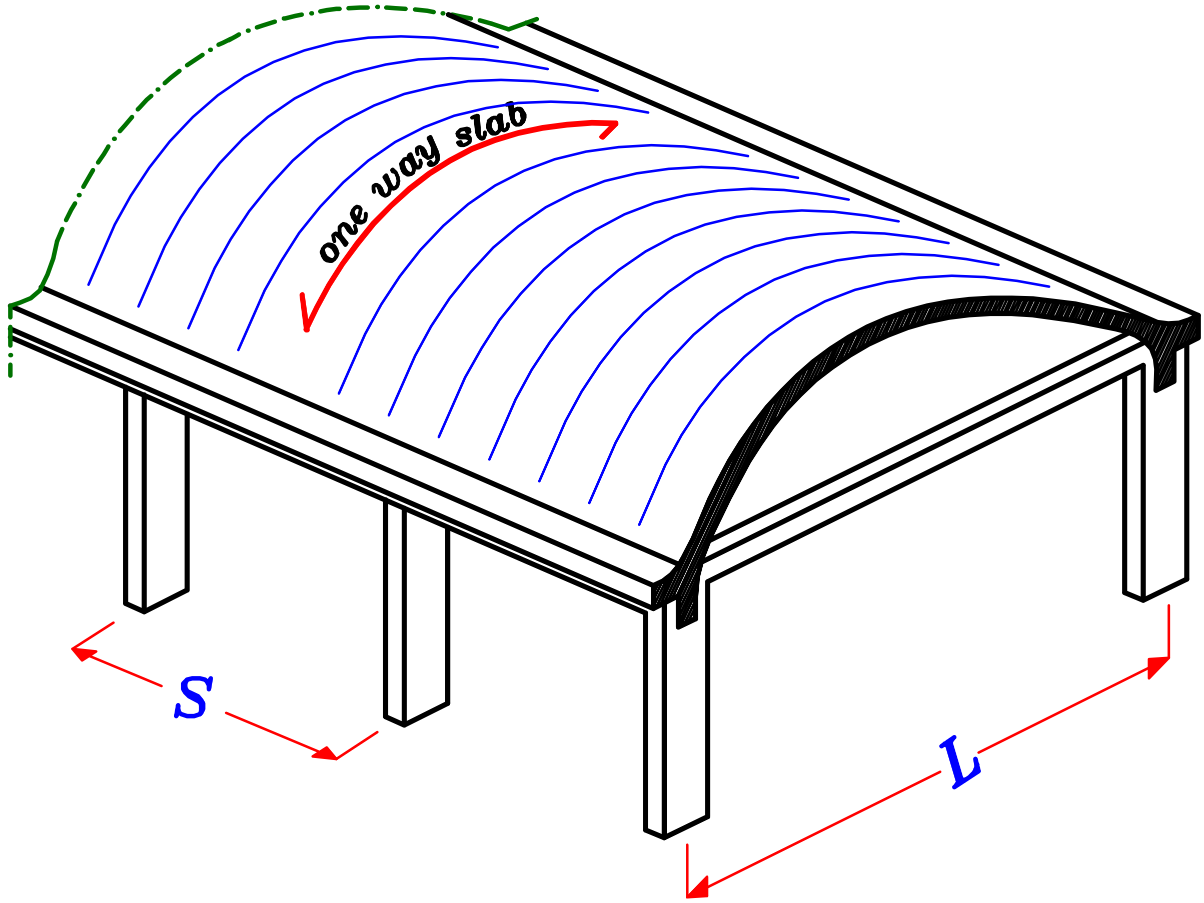


حتى يكون على البلاطه *compression* فقط
و يكون *deflection* البلاطه أقل بكثير
فتكون القطاعات و التسليح أقل و بالتالى أوفر فى ثمن الخامات .

ملحوظه *parabolic slabs* تكون فى الاسطح النهائيه فقط و ليست فى الادوار المتكرره .

ملحوظه

لان الاحمال على الـ *parabolic slab* **قليله** فيكون الـ *tension* على الـ *tie* نسبياً **قليل**
لذلك ممكن للتسهيل اهمال الـ *extension of tie* .

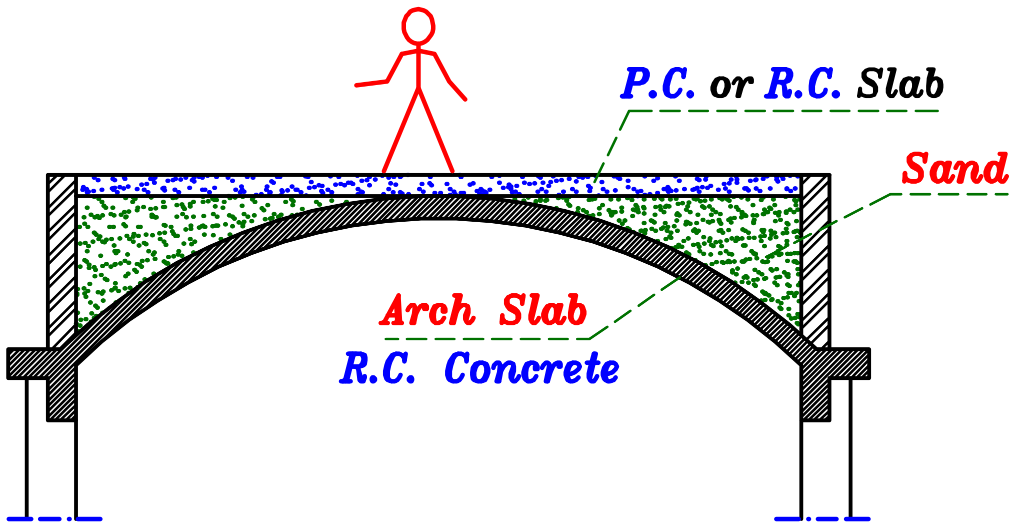


هى عباره عن بلاطه **solid** و تكون **one way** لانها محموله على كمرتين فقط.

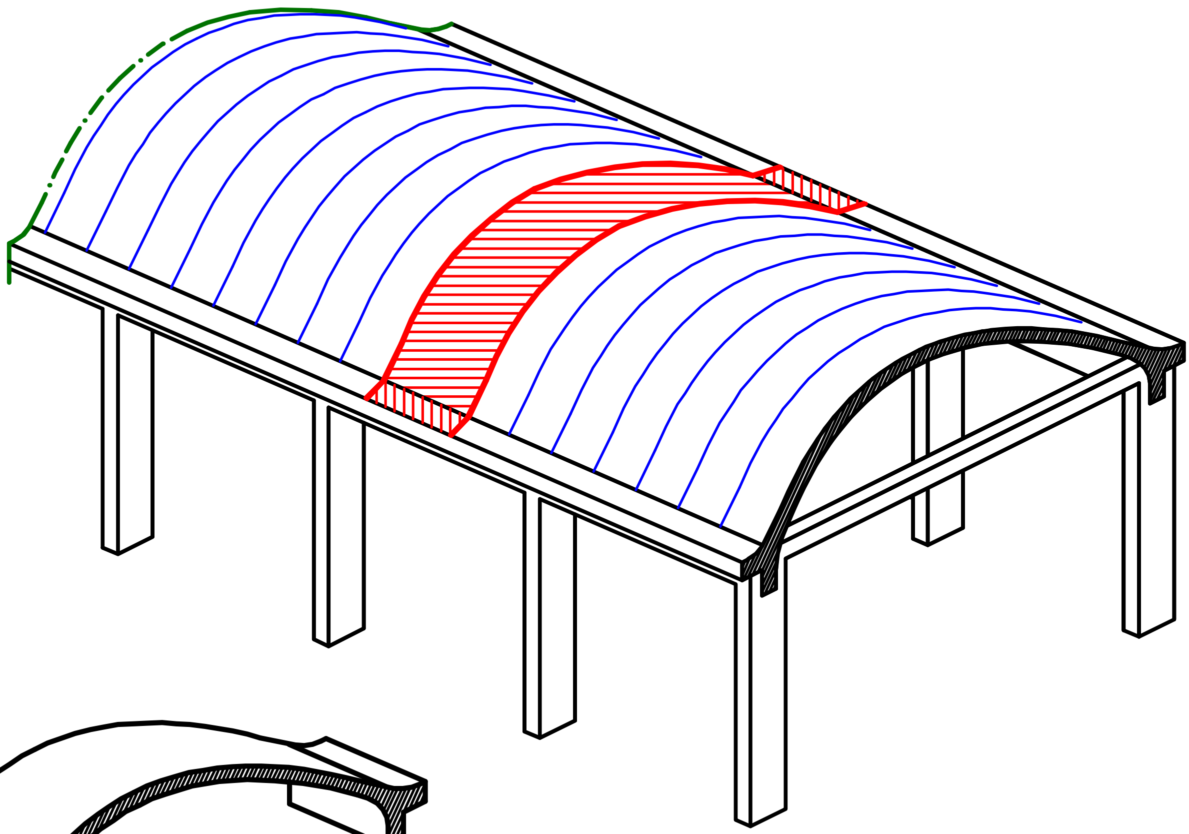
مميزاتها : لان شكلها عكس ال **bending moment**
لا يكون عليها **moment** و يكون عليها **compression Force** فقط
و لا يكون لها **deflection** مما سيؤدى عند التصميم الى ان تكون
كميات الخرسانه و الحديد المطلوبين قليله أى تكون البلاطه أرخص .

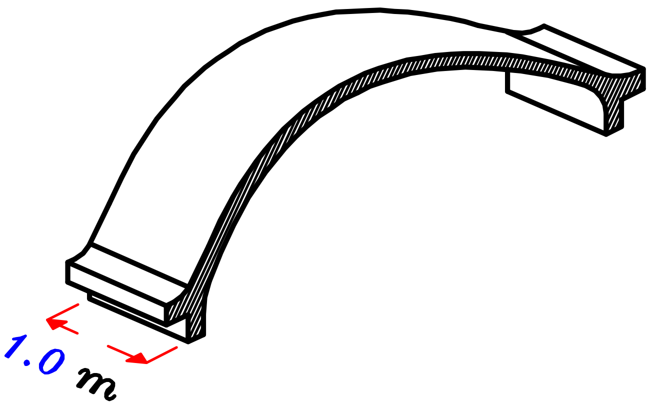
عيوبها : ١ـ تكون الشده فى التنفيذ منحنيه و يكون الحديد منحنى مما يصعب عمليه التنفيذ .
٢ـ يجب أن تكون دور أخير أى لن نستطيع عمل دور فوقها الا بشروط خاصه .

طريقه استخدام ال
parabolic slab
فى حاله الادوار المتكرره

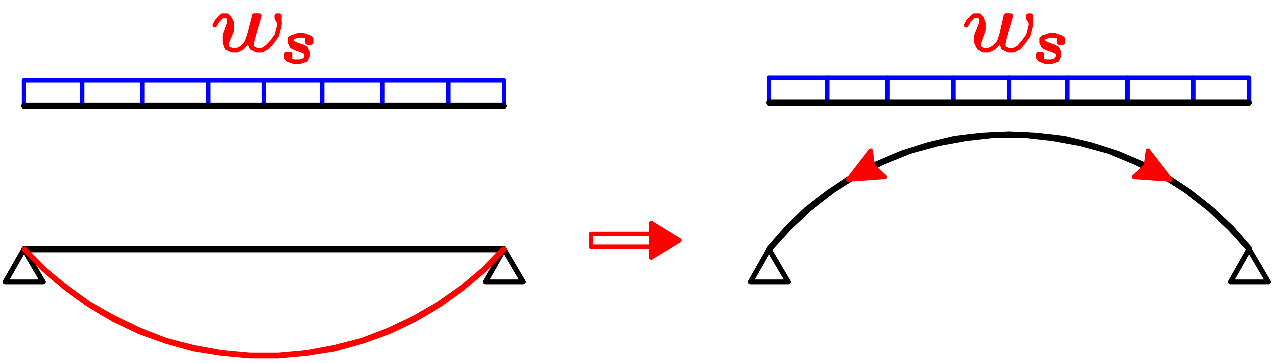


Concept of Parabolic Slab.





بأخذ شريحه فى البلاطه عرضها – ١,٠ م



و لان عاده البلاطات تكون الاحمال عليها **Distributed Loads**

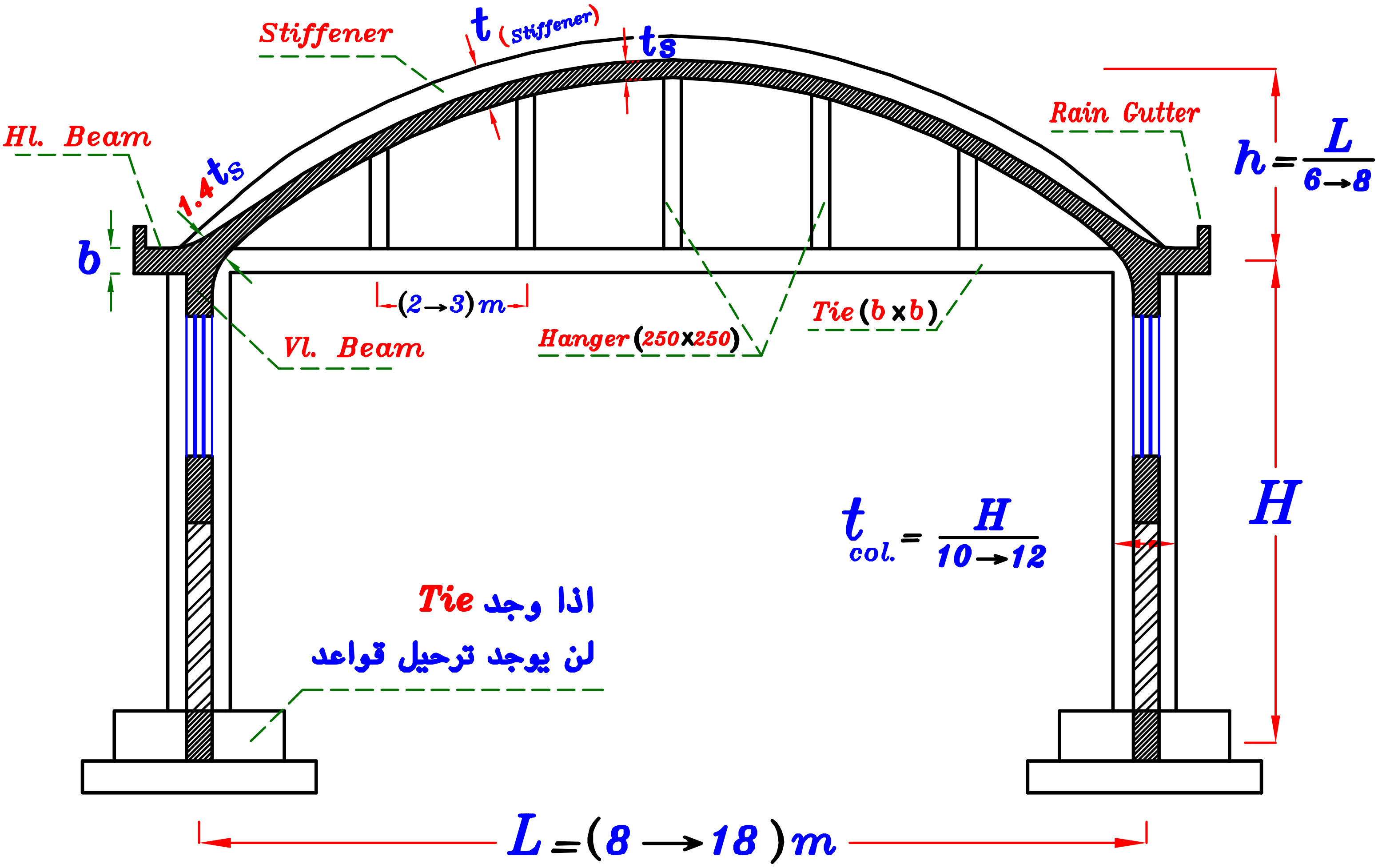
فسيكون ال **moment** المفروض أن يحدث عليها شكله **Parabola** لاسفل

لذا اذا اخذنا شكل البلاطه **Parabola** لاعلى سيكون شكل البلاطه عكس ال **moment**

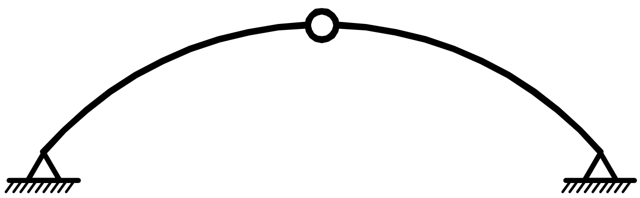
أى أن الشكل الحقيقى للبلاطه يجب ان يكون **Parabola** و ليس **Arch**

أى أن معادلته $\boxed{Y = a X^2 + b X + c}$

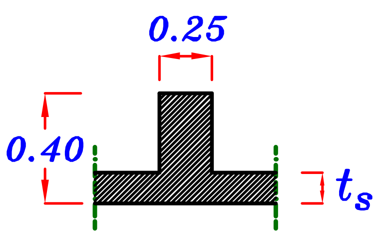
Concrete Dimensions.



* Span (L) = (8 → 18) m
* Hieght (h) = $\frac{L}{6 \to 8}$
* t_s = (8 → 14) cm.
* b = width of HL. Beam
  = (0.25 OR 0.30) m
* Tie (b x b)
* Hanger (250 x 250)
* $t_{col.} = \frac{H}{10 \to 12}$
* Stiffener (250 x 400)



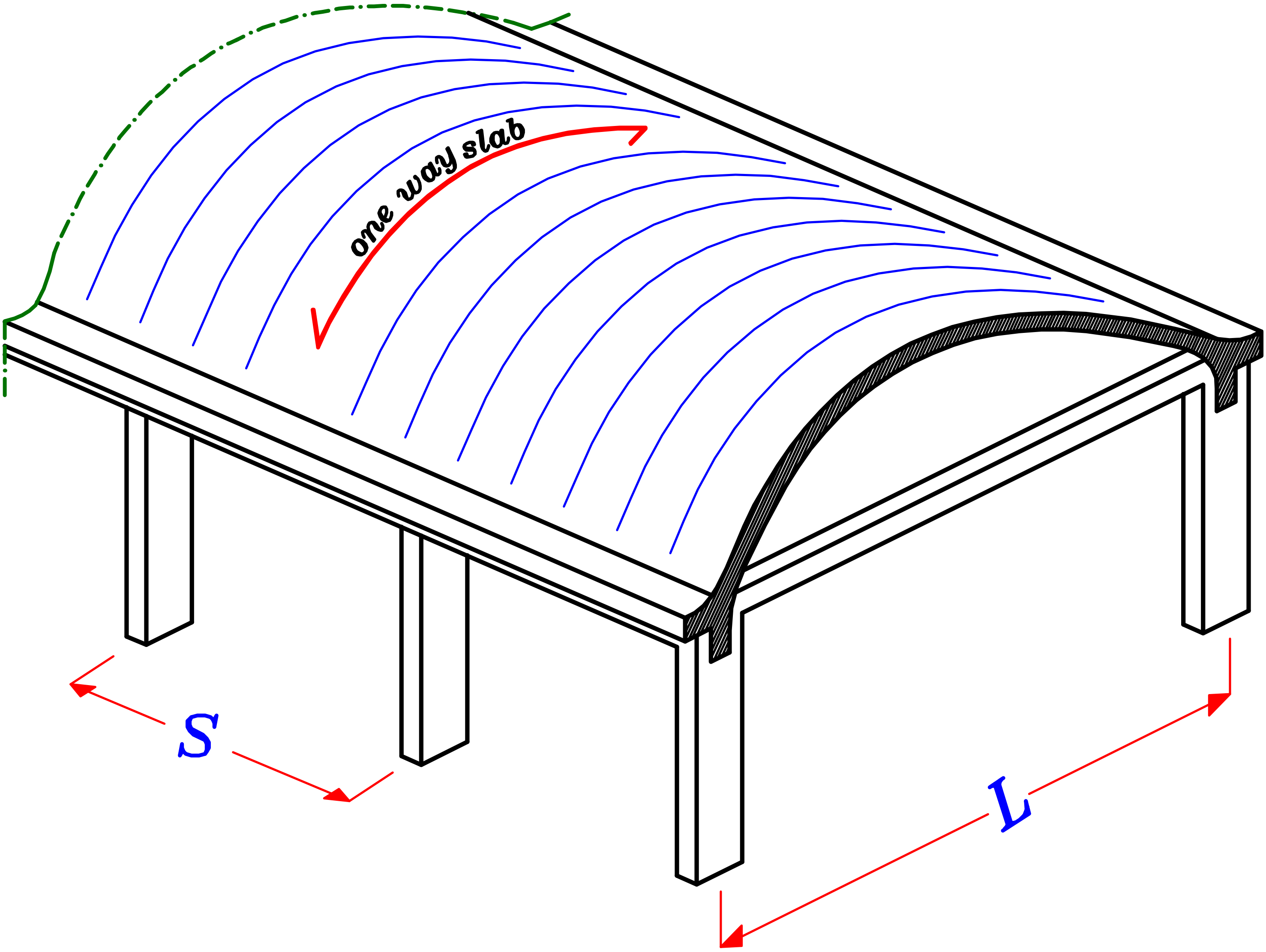
Statical System

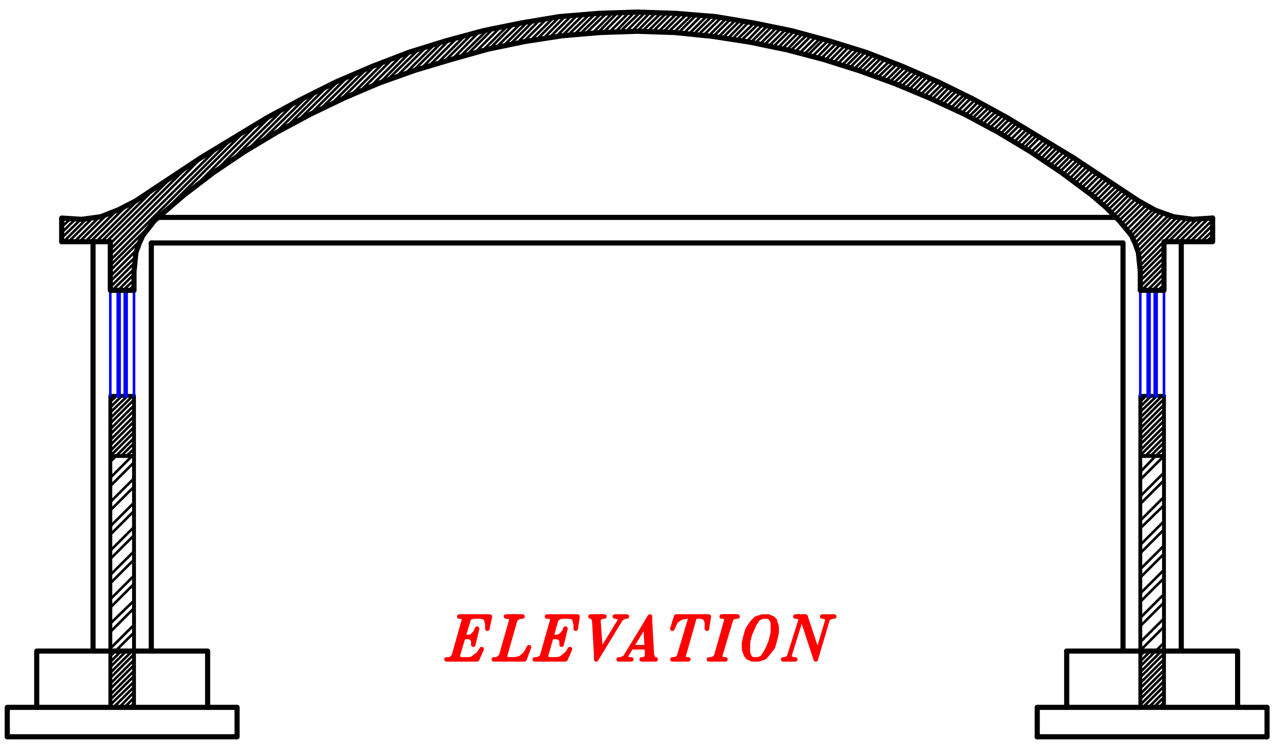


Comp. Force توضع لتقوية البلاطه وتقليل ال Buckling حيث أن البلاطه معرضه ل

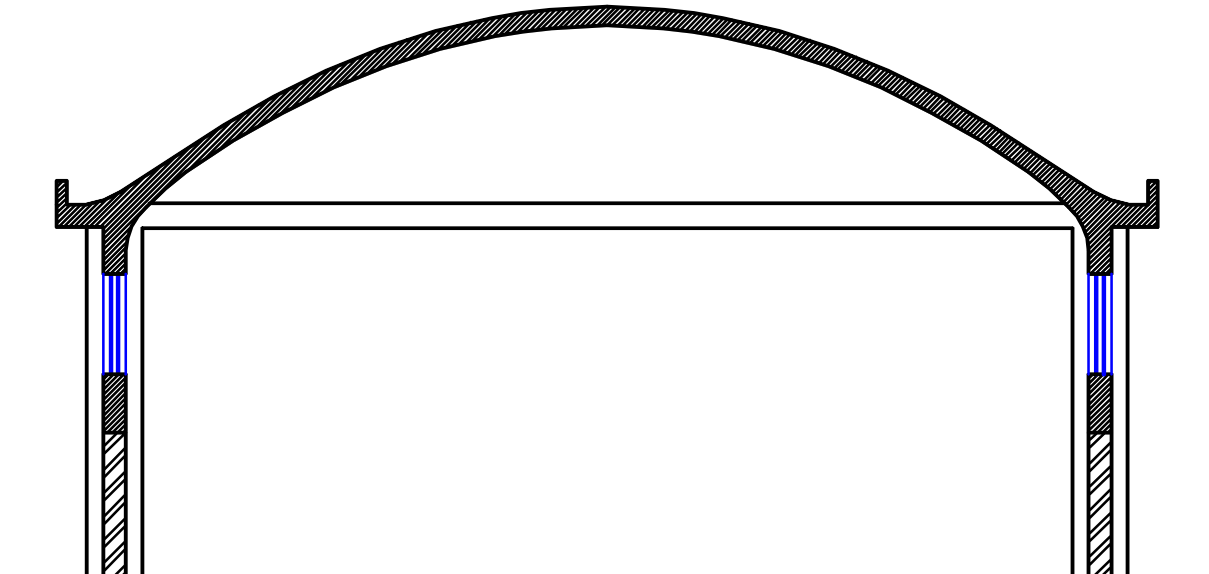
و يفضل وضعها فوق ال Hangers حتى يدخل تسليح ال Hangers بها .

Arch Slab. Without Stiffeners & Without Hangers.

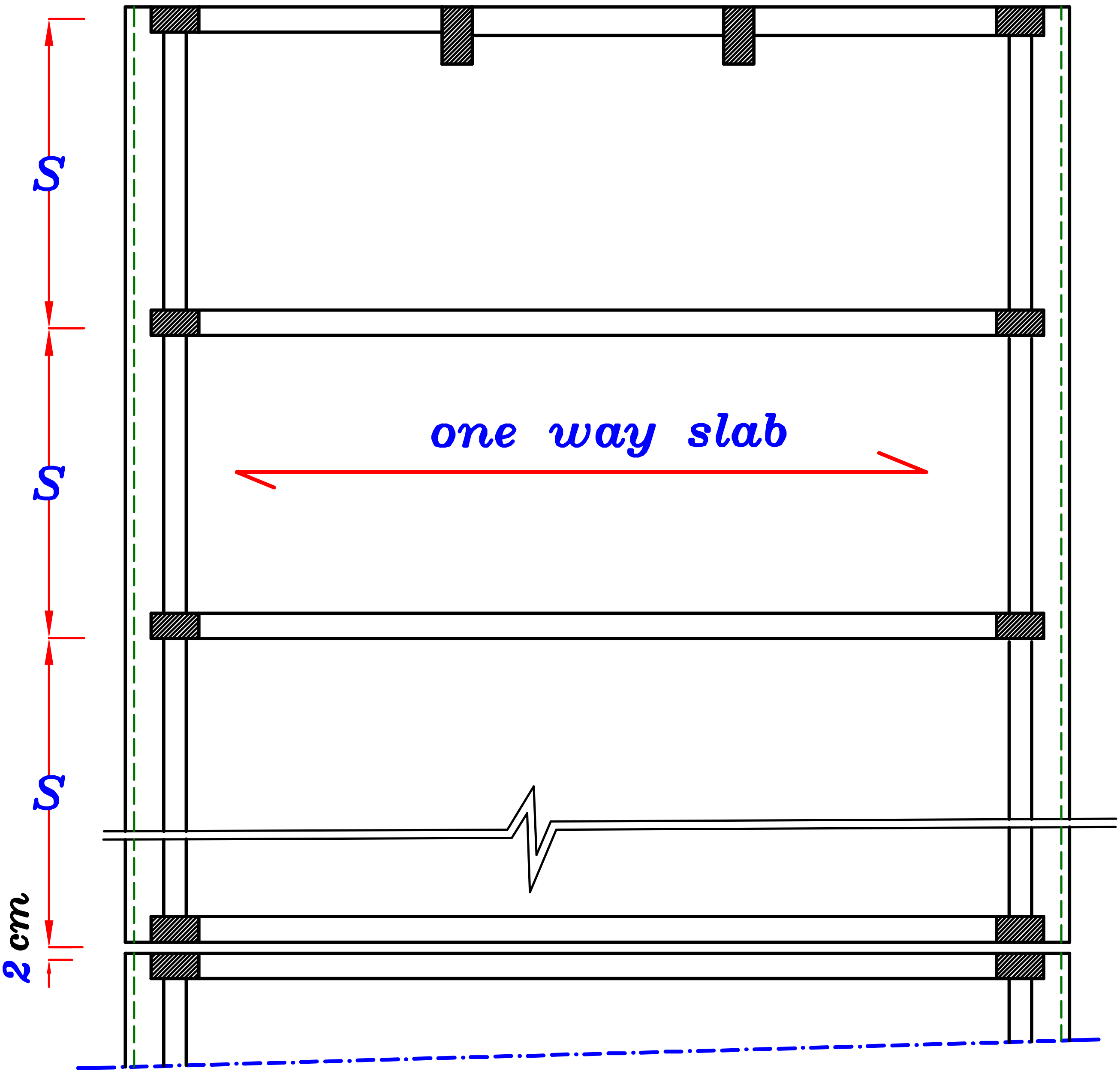




ELEVATION

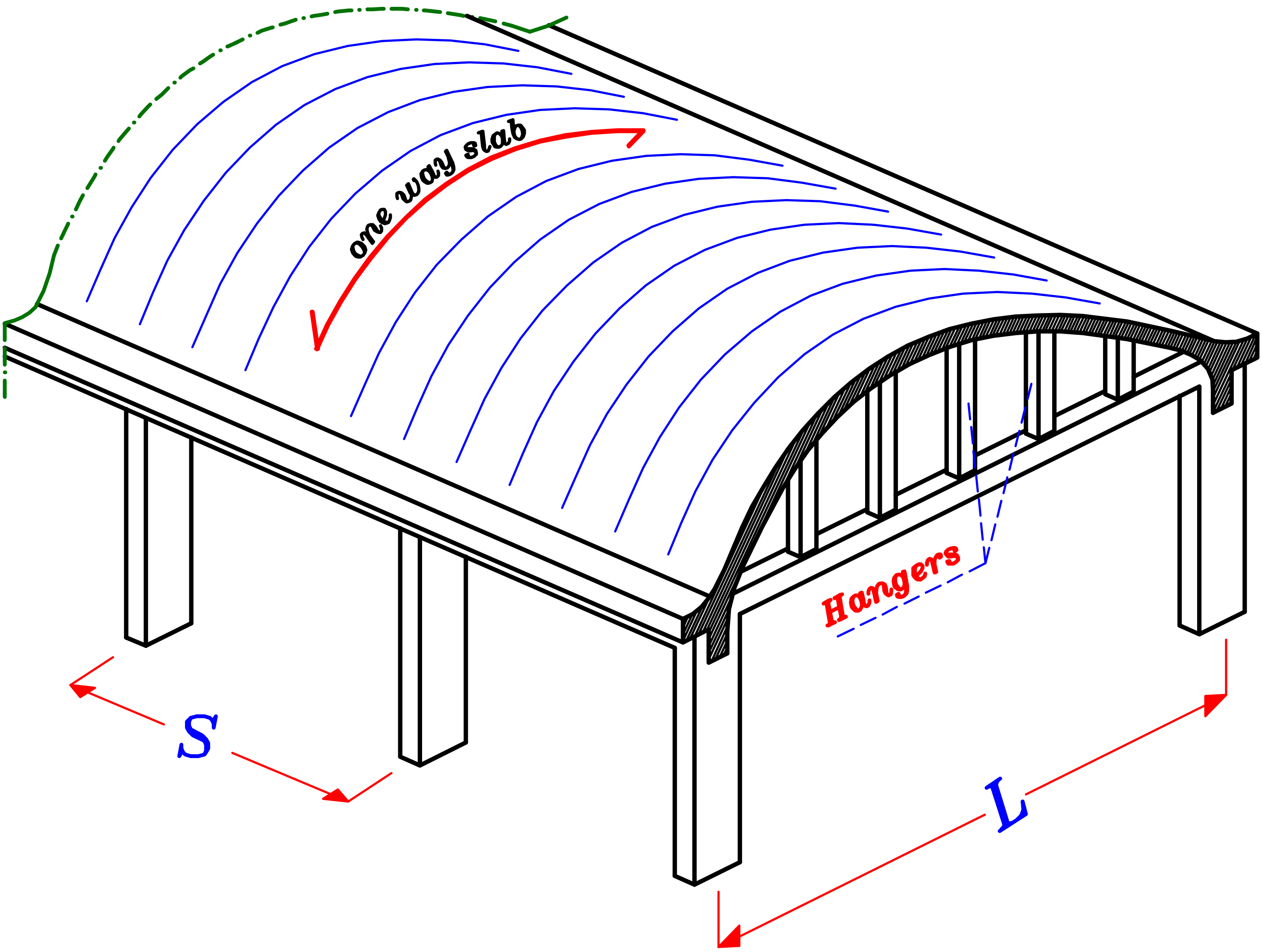


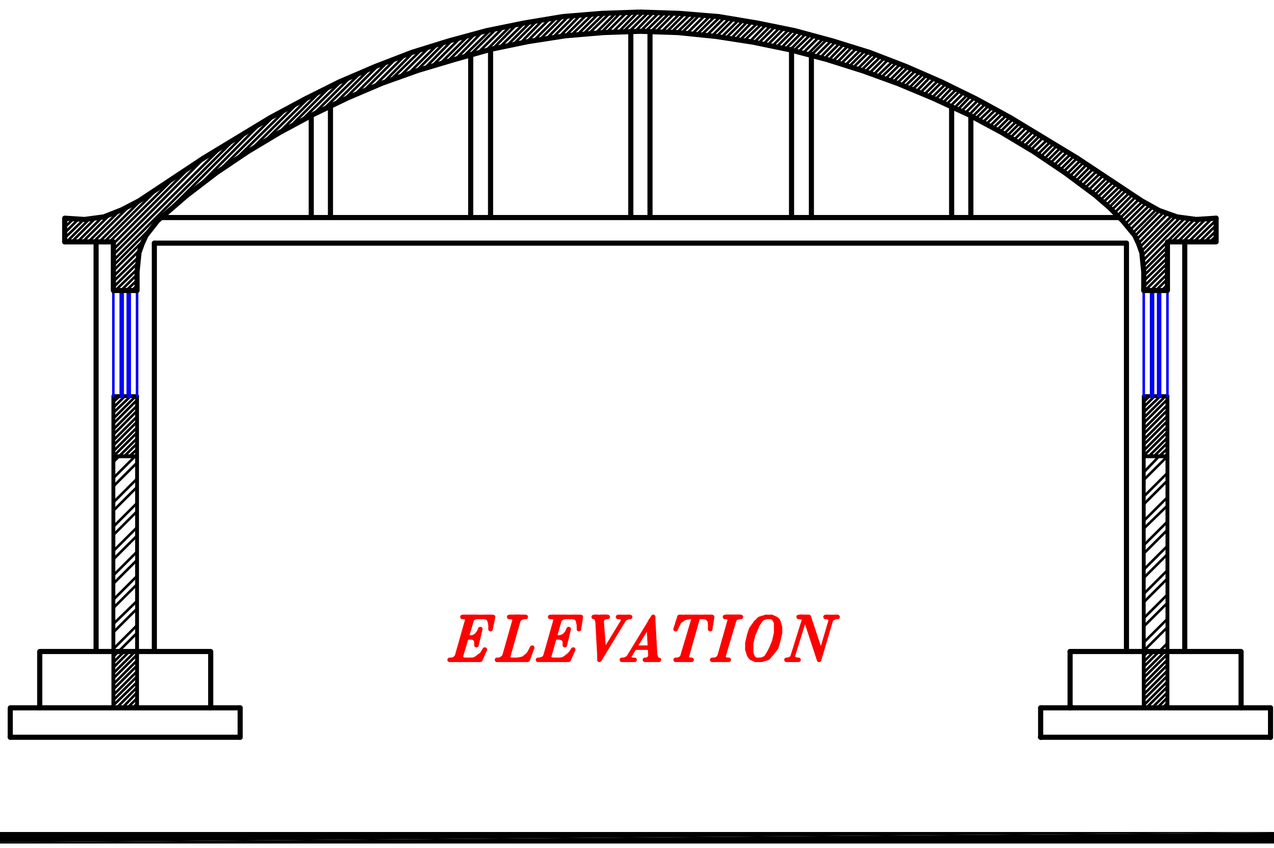
ELEVATION



PLAN

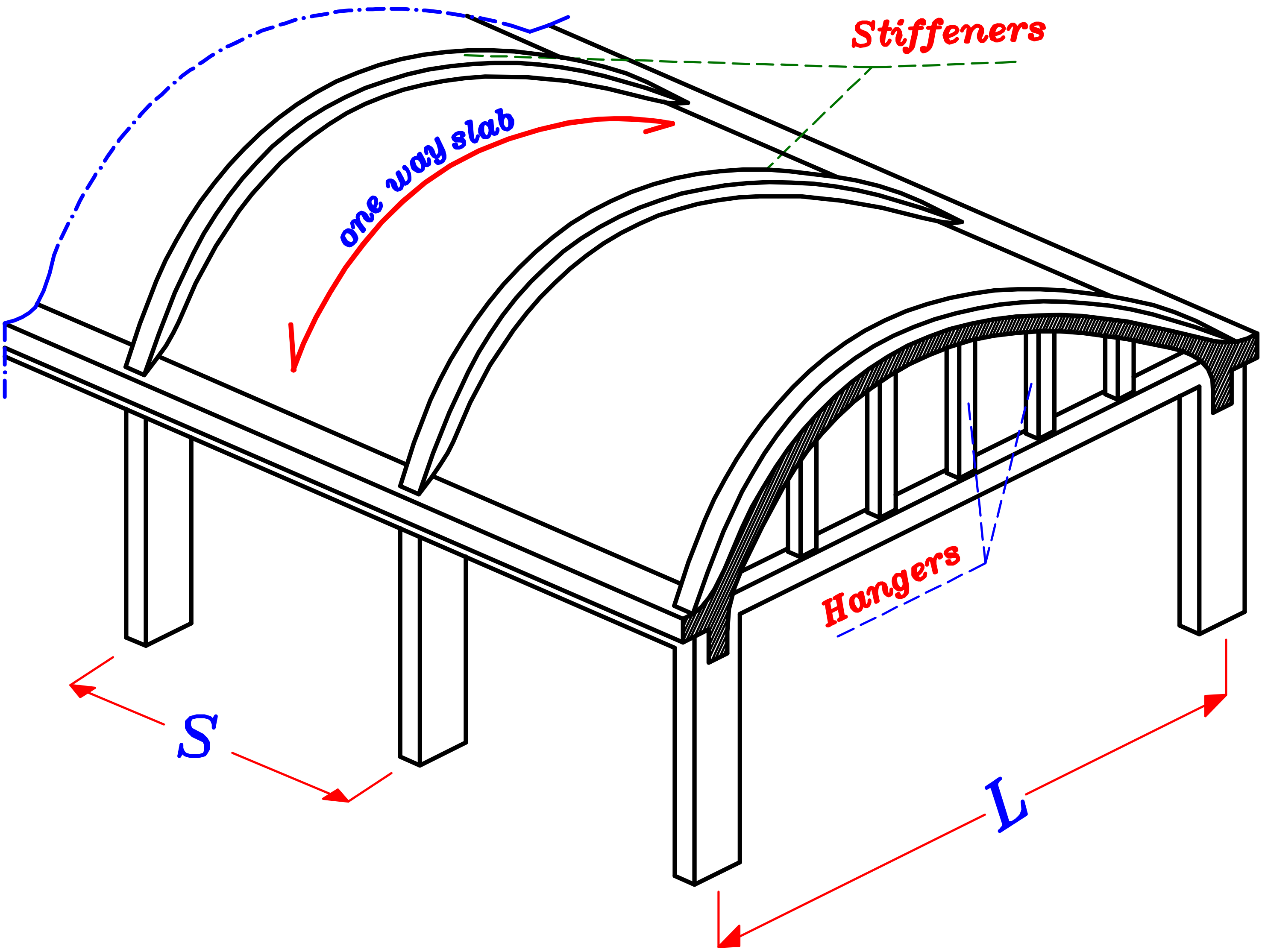
Arch Slab. Without Stiffeners & With Hangers.

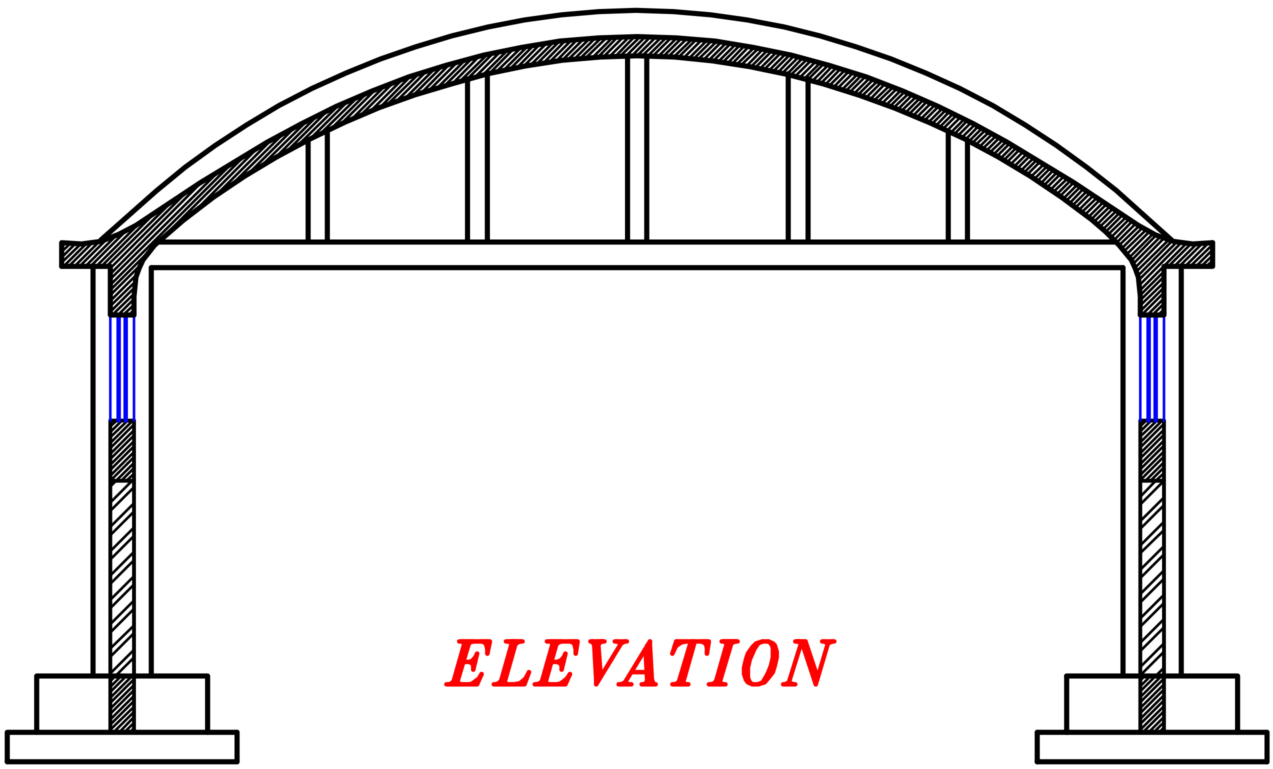




ELEVATION

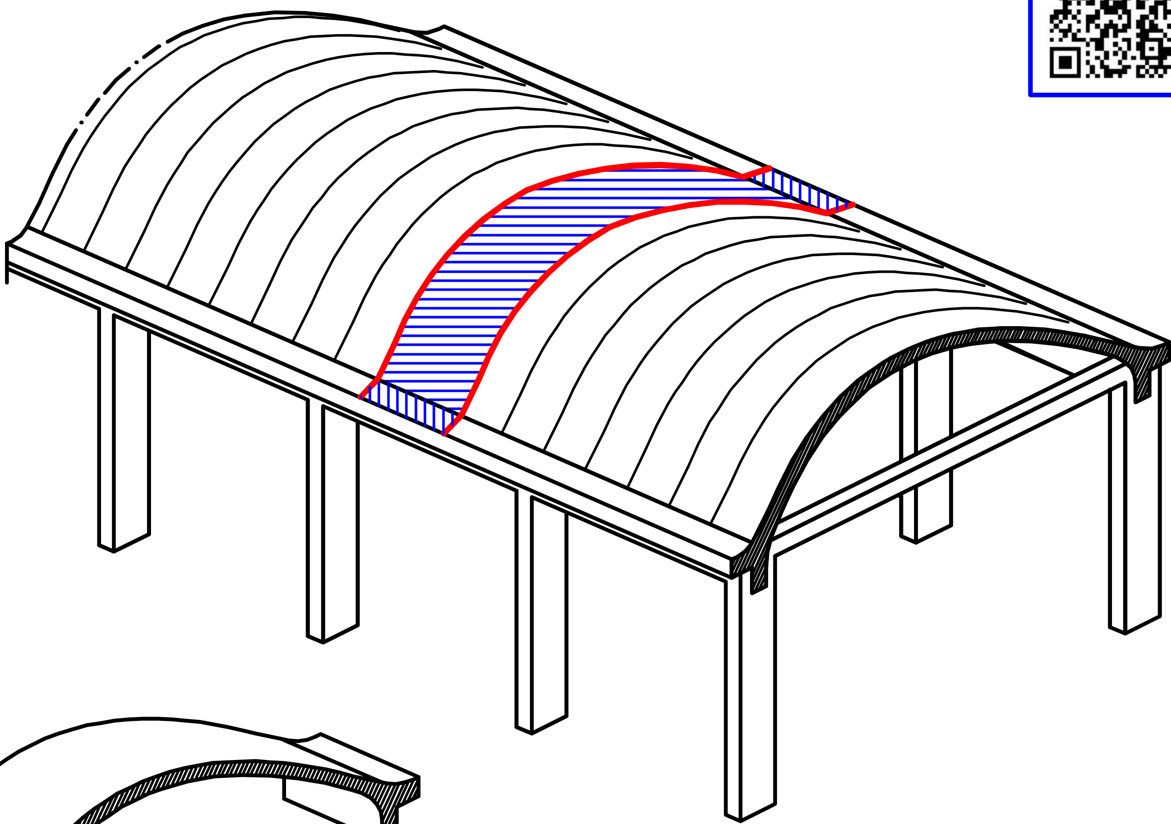
Arch Slab. With Stiffeners & With Hangers.

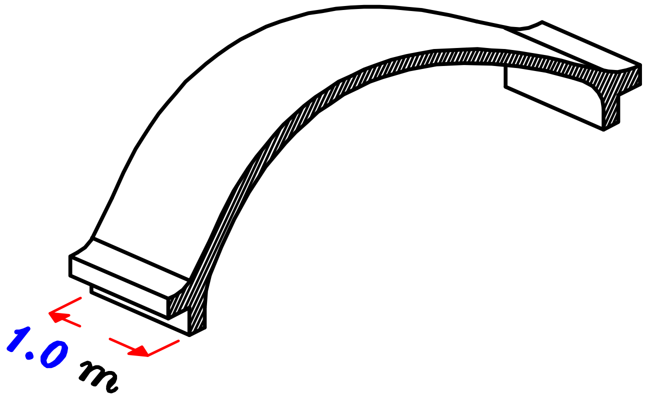




ELEVATION

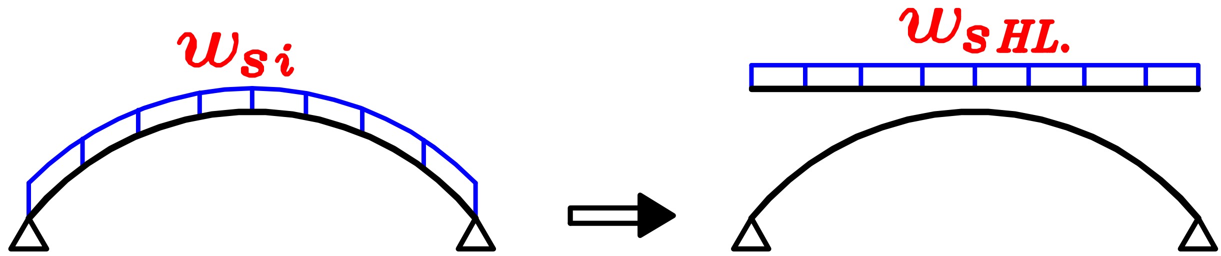
Analysis of Arch Slab.





بأخذ شريحه فى البلاطه عرضها – ١٫٠ م

و نضع عليها حمل منتظم $w_{s\,HL.}$



فى الحسابات يتم وضع الحمل على البلاطه على الافقى و ليس المائل

لاننا لن نستطيع حساب $Cos\,\theta$ لكل نقطه لان قيمه θ تختلف عند كل نقطه .

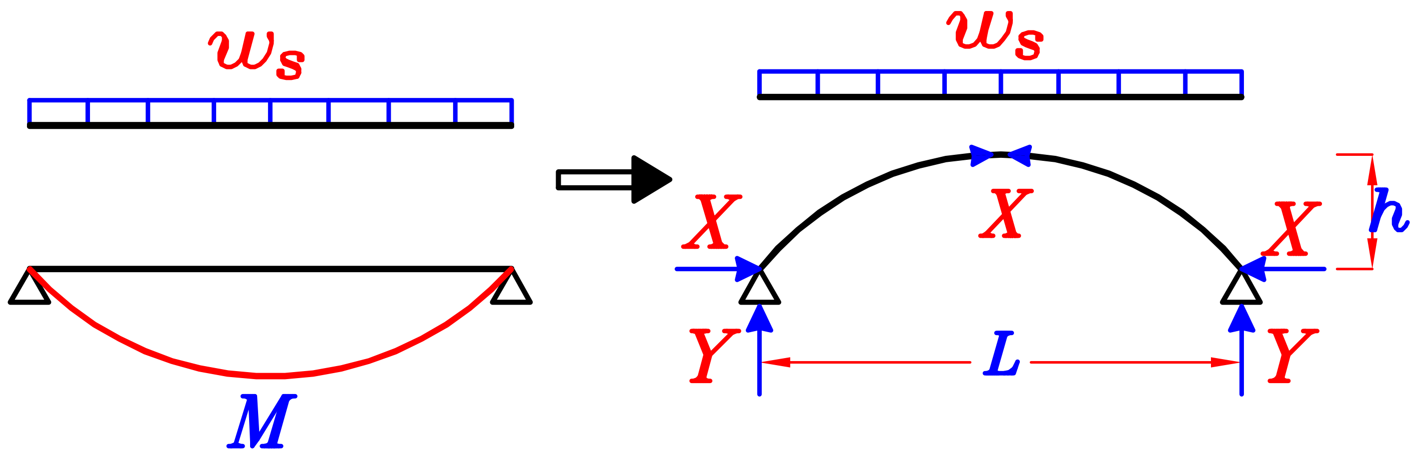
Take $t_s = (100 \rightarrow 140)\ mm$ $\boxed{t_s \simeq 120\ mm}$

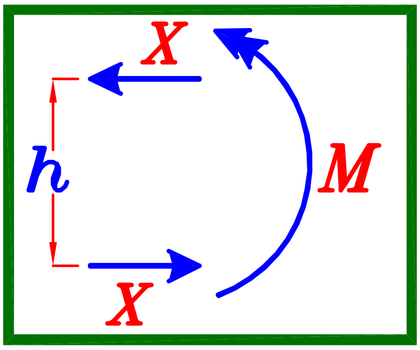
assume $F.C. \simeq 0.50\ kN\backslash m^2$, $L.L. \simeq 0.50\ kN\backslash m^2$

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.) \simeq 5.0\ kN\backslash m^2$$

تعتمد الفكره على تحويل ال Bending moment الي Couple

اى الى Compression Normal Forces & Tension Normal Forces





$$Y = \frac{w_s L}{2}$$

$$X = \frac{M}{h}$$

لان الاحمال على البلاطه المنحنيه تعتبر صغيره فبالتالى ستكون قيمه X صغيره

فاذا وضعنا Tie حتى تقاوم قيمه X فلن يكون عليها Tension كبير

و فى هذه الحالة ممكن اهمال ال Extension of Tie

Get N.F. & B.M.

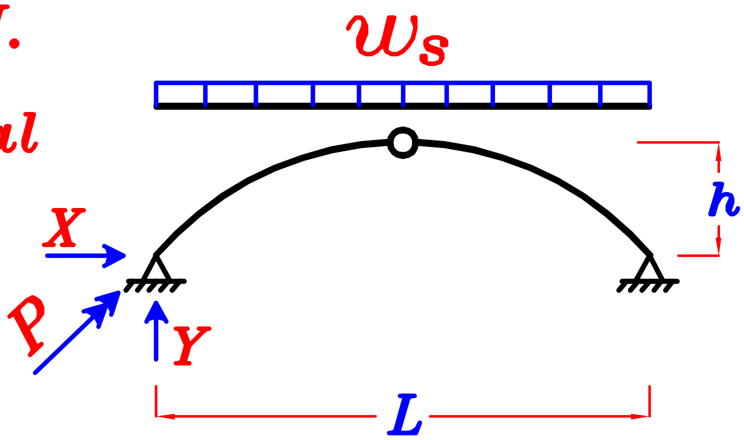
To get max Normal

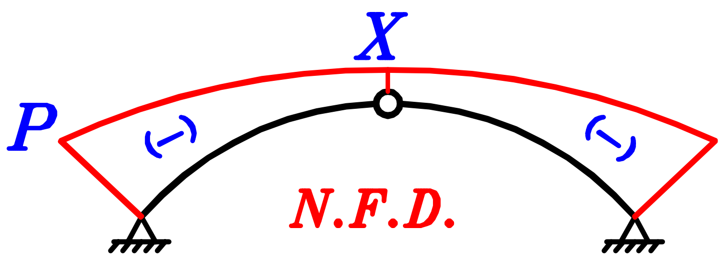
B.M. = Zero

$$Y = \frac{\sum Loads}{2} = \frac{w_s L}{2}$$

$$X = \frac{M}{h} = \frac{w_s L^2}{8h}$$

$$P = \sqrt{X^2 + Y^2}$$

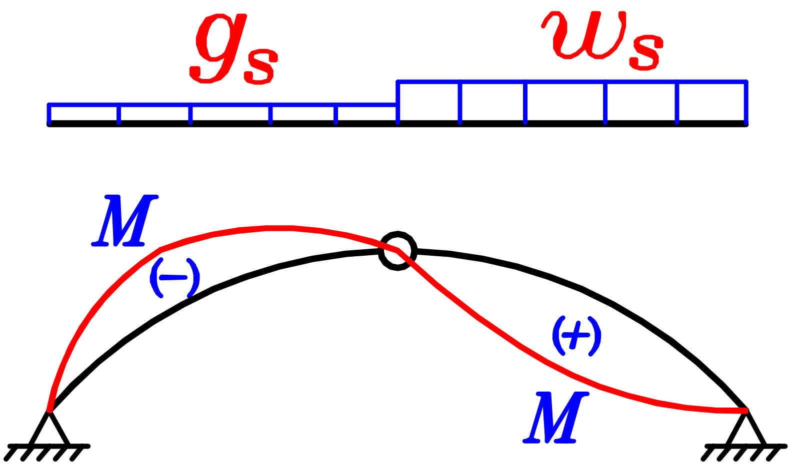




To get max B.M.

لعمل bending moment على البلاطه

نعمل حالات تحميل

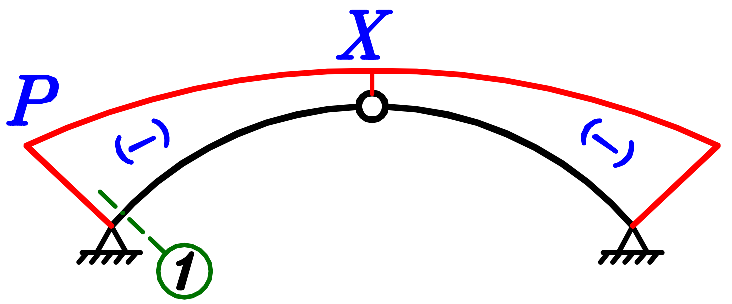


$$\boxed{M = \frac{p_s * L^2}{64}}$$

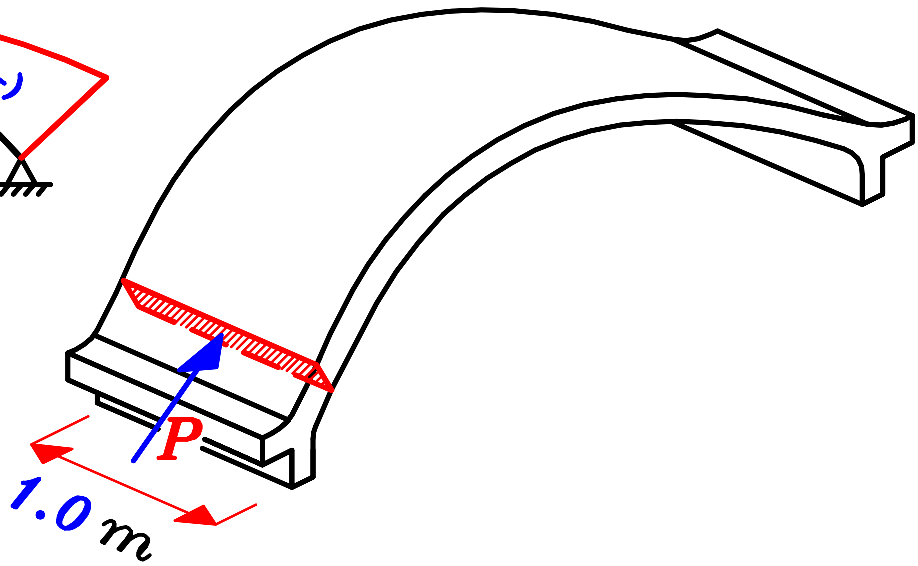
where : $p_s = w_s - g_s$

قيمه صغيره جدا جدا ممكن اهمالها

Design Critical Section of Arch Slab.

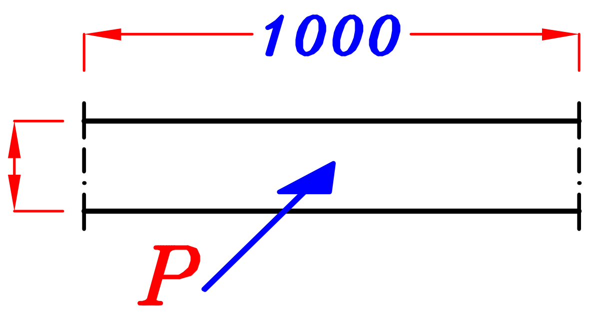


$$P = \sqrt{X^2 + Y^2}$$



Design on N.F. only.

Sec (1−1)



$$\therefore \boxed{P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y}$$

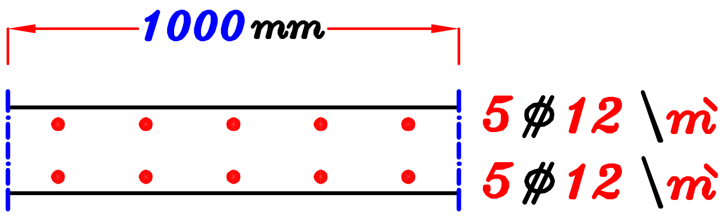
$P_{U.L.} = P$, $A_c = t_s * 1000 \longrightarrow$ Get $A_s = \checkmark\ mm^2$

عاده تكون A_s أقل من $A_{s\,min}$

$$\therefore \text{ Take } A_s = A_{s\,min} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 120 * 1000$$

$= 960\ mm^2 \simeq 10\phi 12 \backslash m$ مجموع الحديد السفلى و العلوى

$$\boxed{A_s = A_s` \simeq 5\phi 12 \backslash m}$$



Drawing Arch Slab.

لان شكل الـ Arch Slab فى الحقيقه عباره عن Parabola

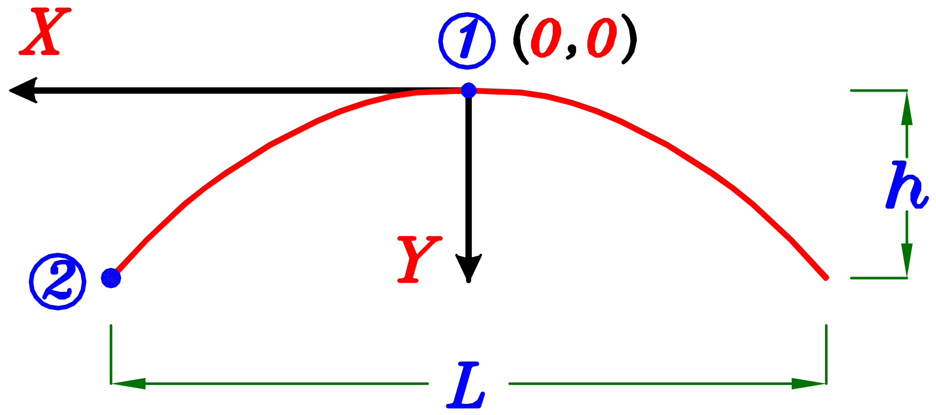
لذا لرسم منحنى الـ Parabola توجد طريقتين :

① By using Equations.

لان معادله الـ Parabola $\boxed{Y = a X^2 + b X + c}$

و لكن اذا اخذنا اعلى نقطه فى البلاطه هى نقطه (0,0)

ستتحول المعادله الى $\boxed{Y = a X^2}$



لتحديد قيمه a

بالتعويض فى النقطه ② $\boxed{X = \frac{L}{2}}$, $\boxed{Y = h}$

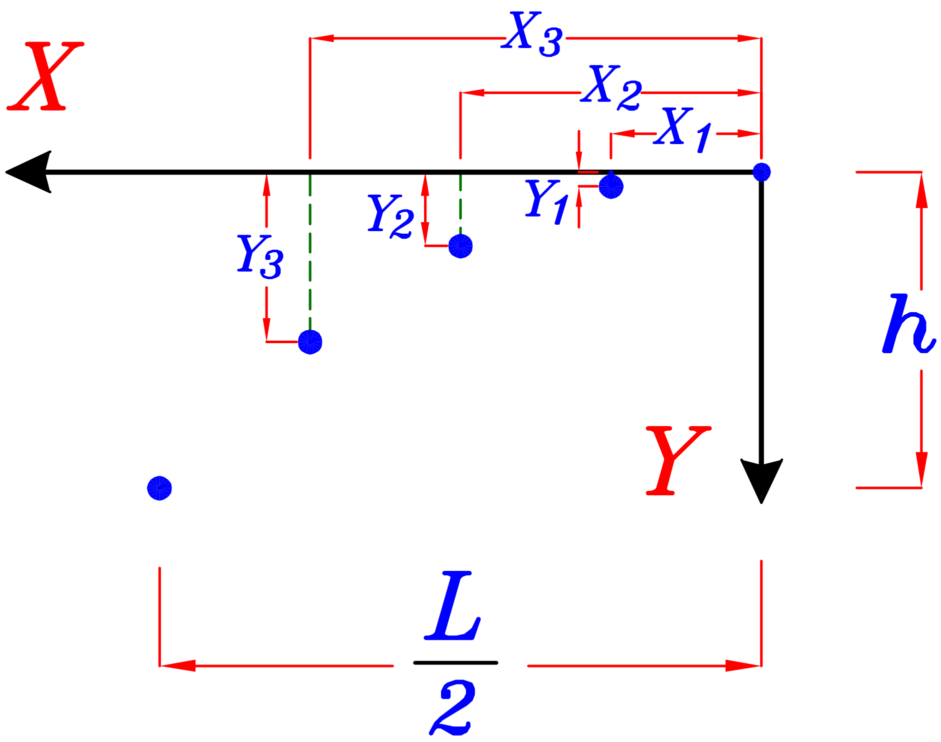
$$Y = a X^2 \longrightarrow h = a \left(\frac{L}{2}\right)^2 \longrightarrow \boxed{a = \frac{4h}{L^2}}$$

$$\therefore \quad \boxed{Y = \frac{4h}{L^2} * X^2}$$

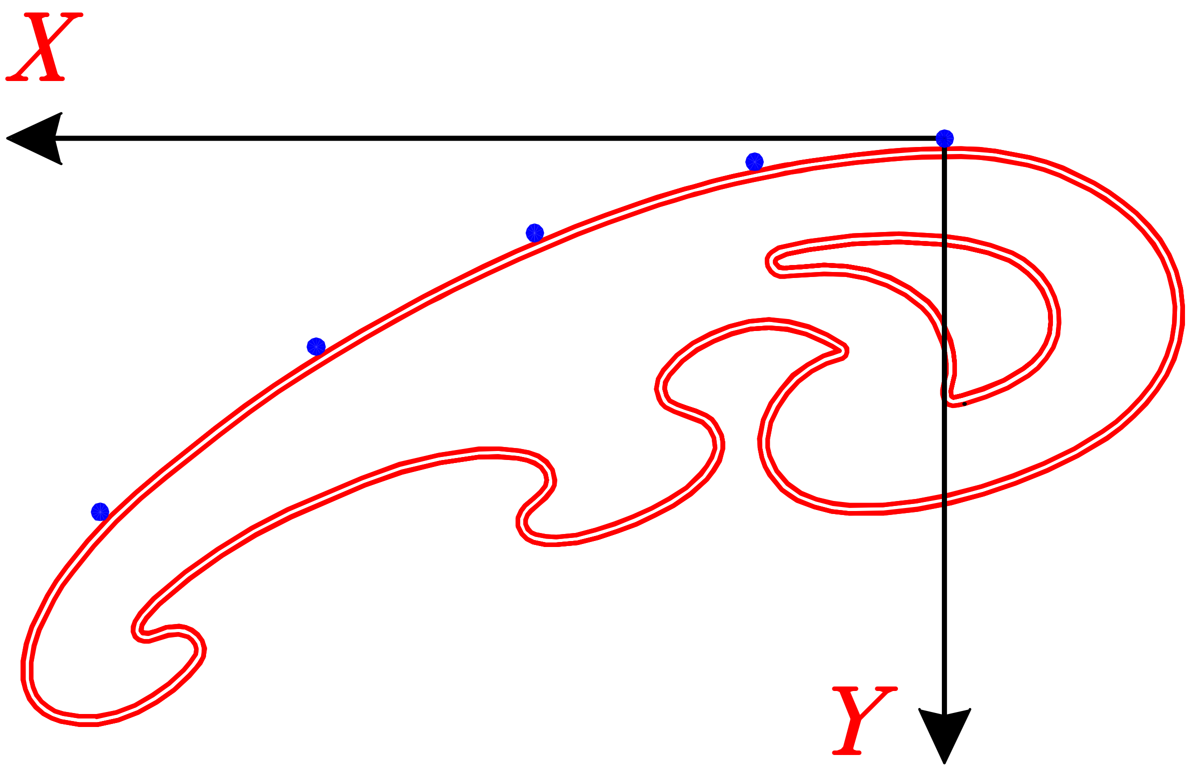
$$\boxed{Y = \frac{4\,h}{L^2} * X^2}$$

بالتعويض فى المعادله عند عده نقط

نفرض قيمه X ثم نحسب لها ال Y



سيكون لدينا عده نقاط على المنحنى ممكن التوصيل بينهم بال ***French Curve***

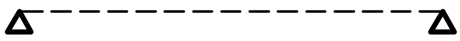


② *By Graphical Method.*

ممكن عمل ٧ مماسات و رسم *curve* يمسهم جميعا .

١- نرسم الخط الافقى للبلاطه

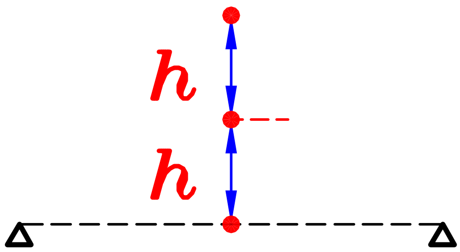
①



٢- نرسم قيمه *h* للبلاطه

ثم نرسم مسافه أخرى بنفس القيمه .

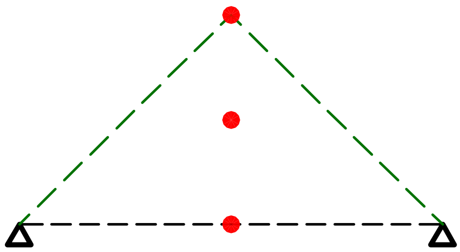
②



٣- نوصل من النقطه العليا الى بدايه

و نهايه ال *parabola*

③

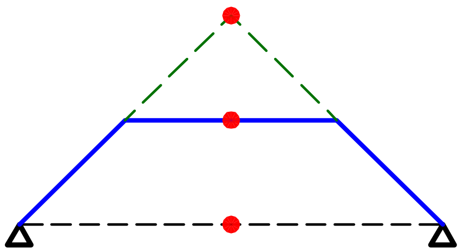


٤- نوصل خط أفقى من النقطه التى فى المنتصف

موازى لل *datum*

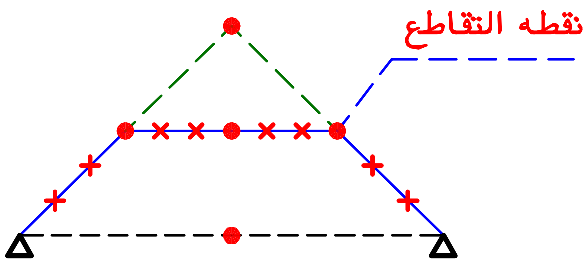
فيتكون ثلاث مماسات لل*parabola*

④



٥- نقسم كل خط الى ثلاث مسافات متساويه

⑤

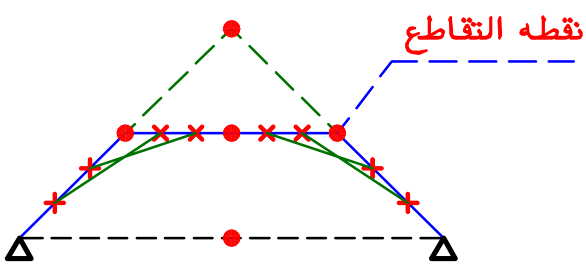


٦- نوصل خطوط بين النقط بحيث نوصل النقطه

القريبه من نقطه التقاطع بالنقطه البعيده

فيتكون ال ٧ مماسات لل *parabola*

⑥

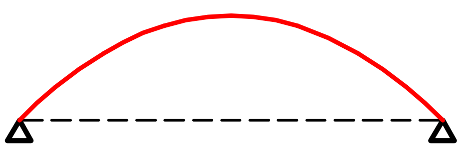


٧- يتم عمل *curve* يرسم بال *French curve*

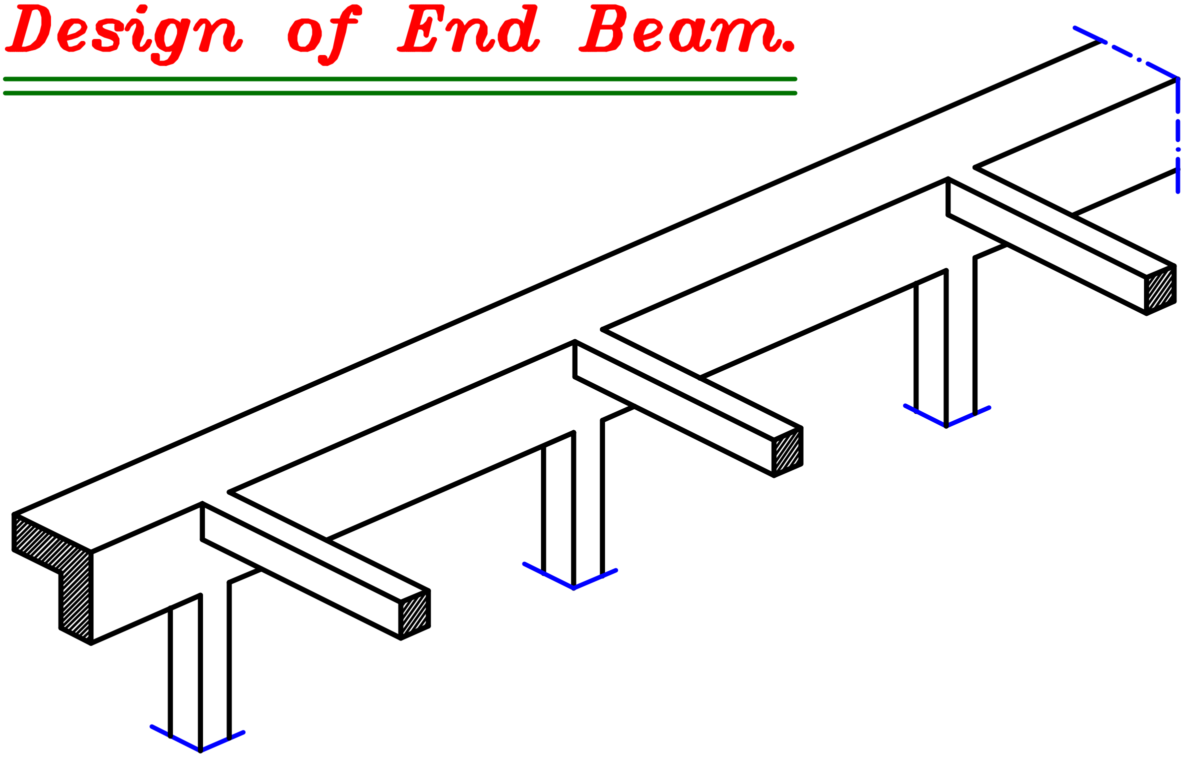
يمس السبع مماسات فيكون هو ال *parabola*

المطلوب لل *moment*

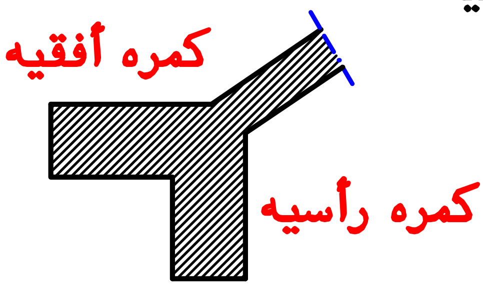
⑦



* Design of End Beam.



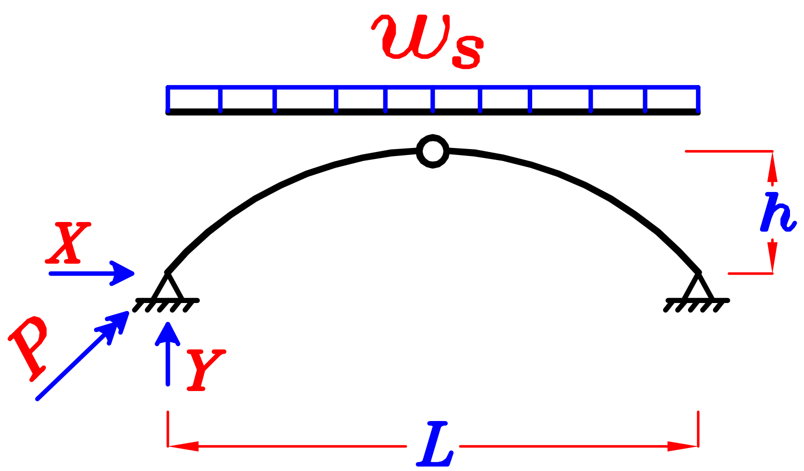
- الكمره الطرفيه *End beam* يوجد عليها قوه أفقيه
لذا تتكون من كمرتين
كمره رأسيه لتحمل الاحمال الرأسيه
كمره أفقيه لتحمل الاحمال الافقيه .



- أى قوى رأسيه تذهب الى الكمره الرأسيه
أى قوى أفقيه تذهب الى الكمره الافقيه .

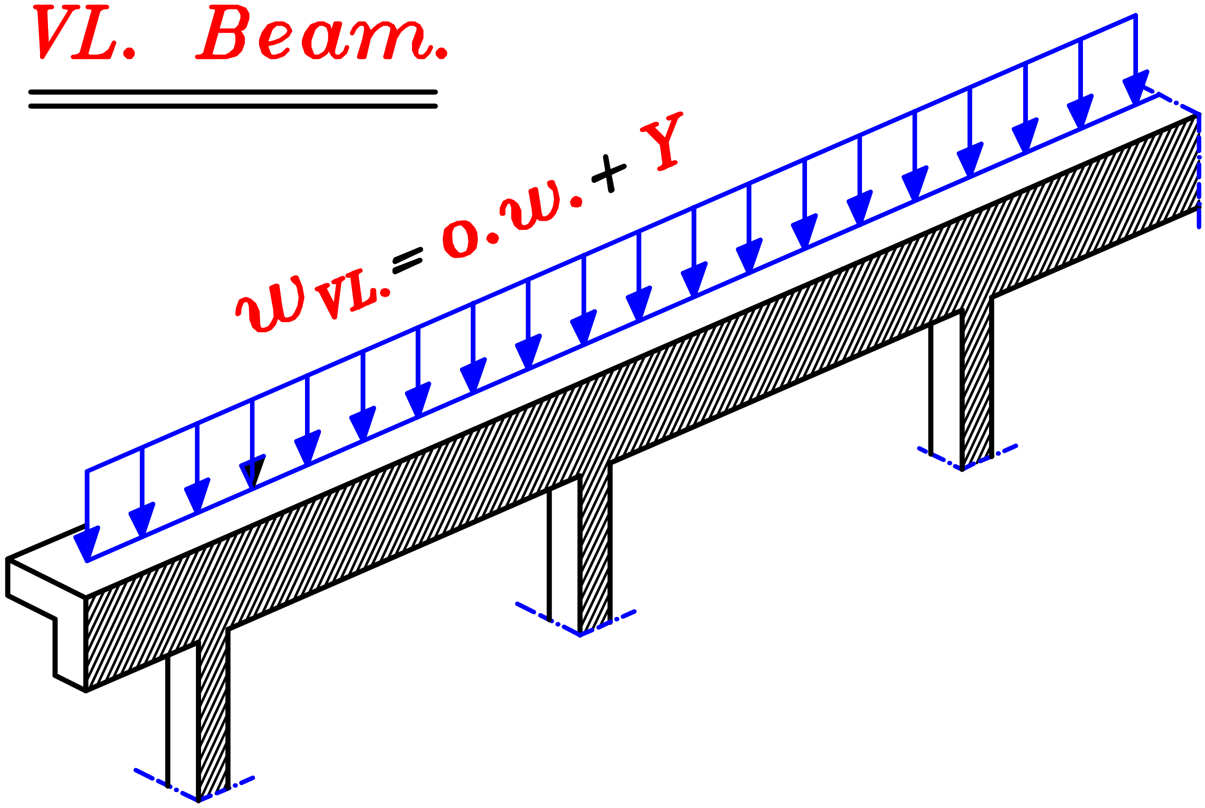
- وزن الكمرتين هو حمل رأسى لذا يذهب الى الكمره الرأسيه فقط.

$$\boxed{O.W._{(beam)}\ (VL.+HL.) \simeq 7.0\ kN/m}$$



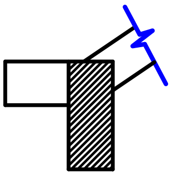
- X, Y من شريحه البلاطه
تنتقل على الـ *End beam*
Y تذهب الى الكمره الرأسيه .
X تذهب الى الكمره الافقيه .

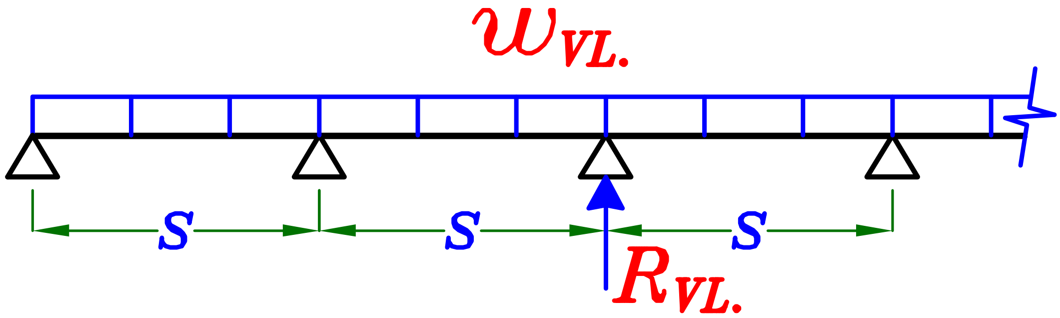
VL. Beam.

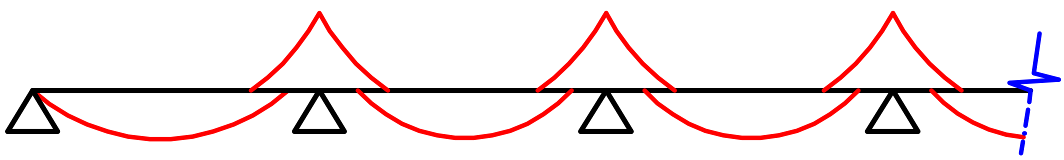


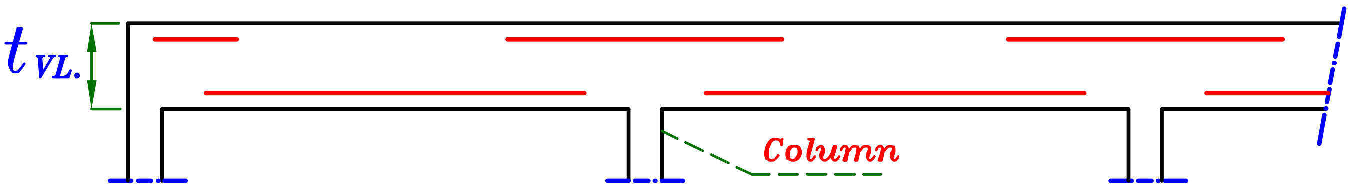
$$w_{VL.} = O.W._{(beam)} + Y \quad kN\backslash m'$$

Designed as R-Sec.





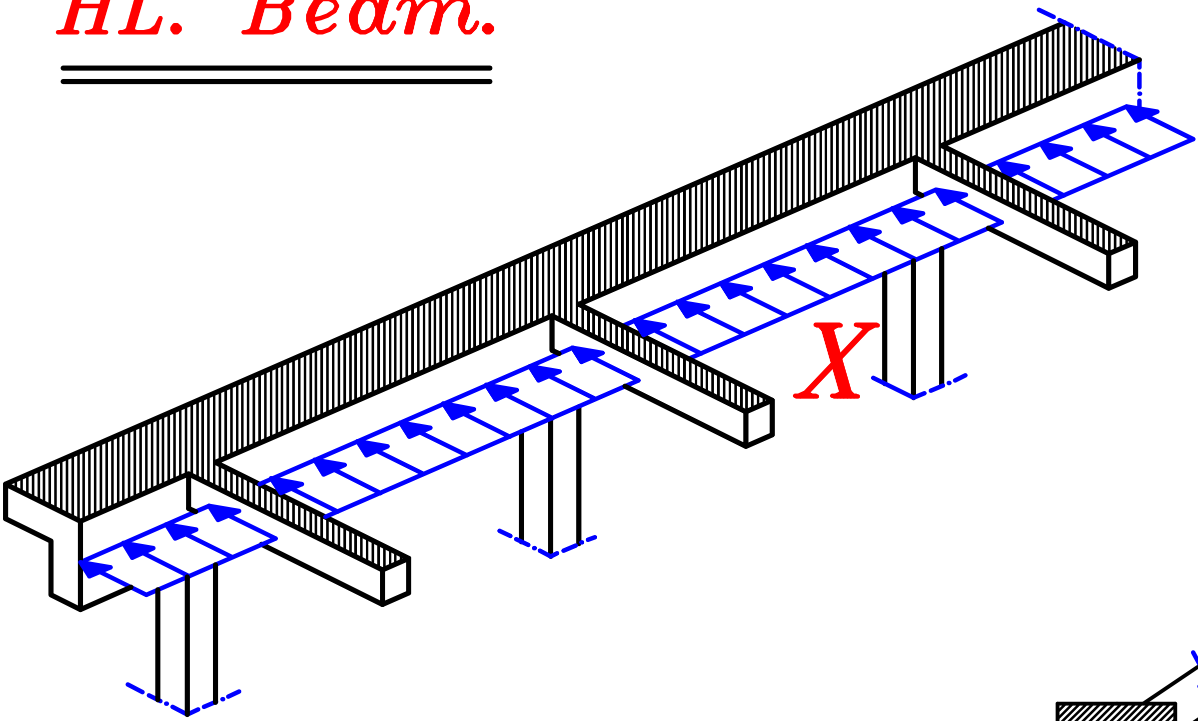




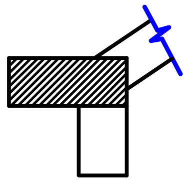
$$\boxed{R_{VL.} = (o.w. + Y) * S}$$

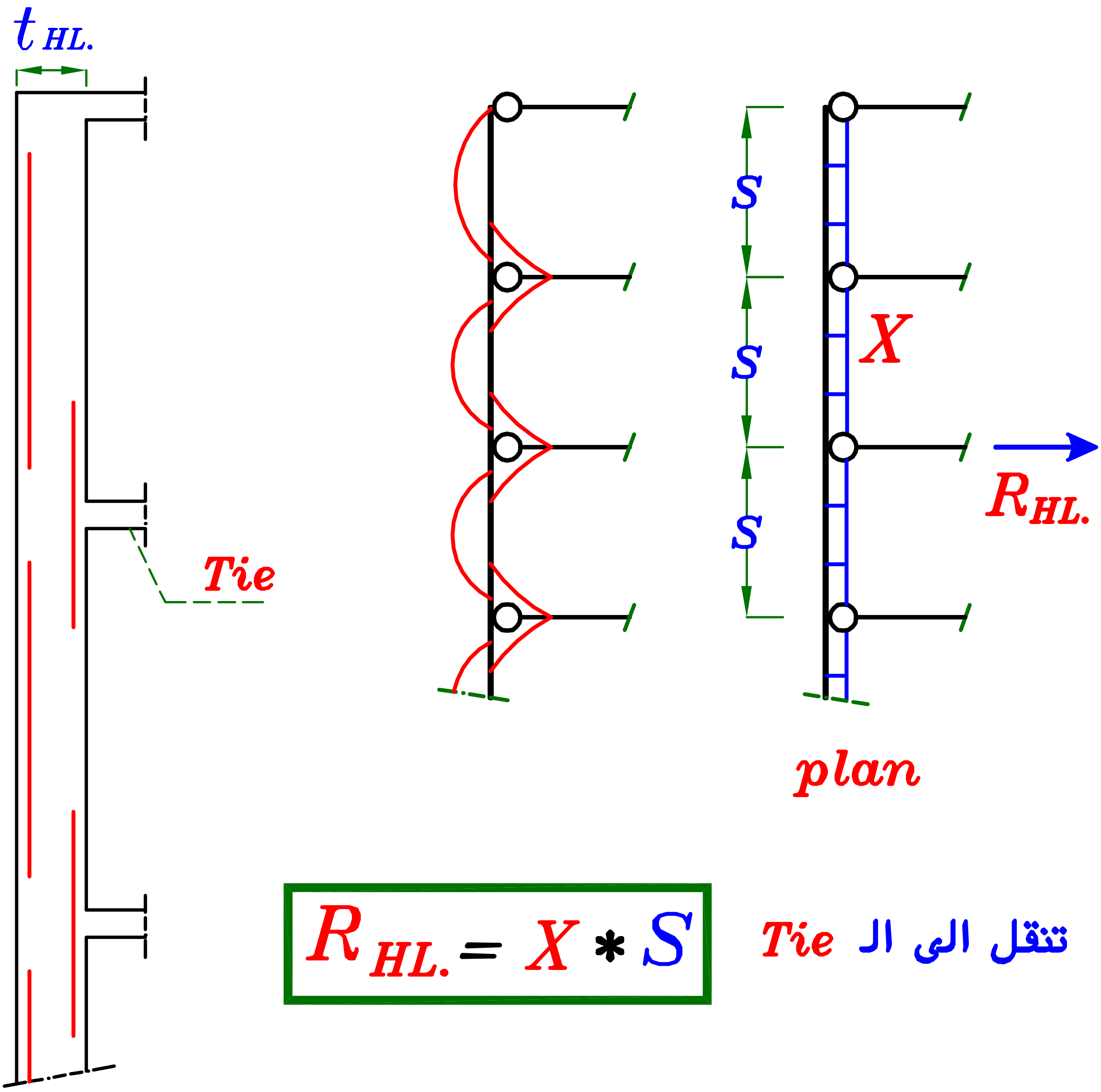
تنقل الى العمود

HL. Beam.



Designed as R-Sec.

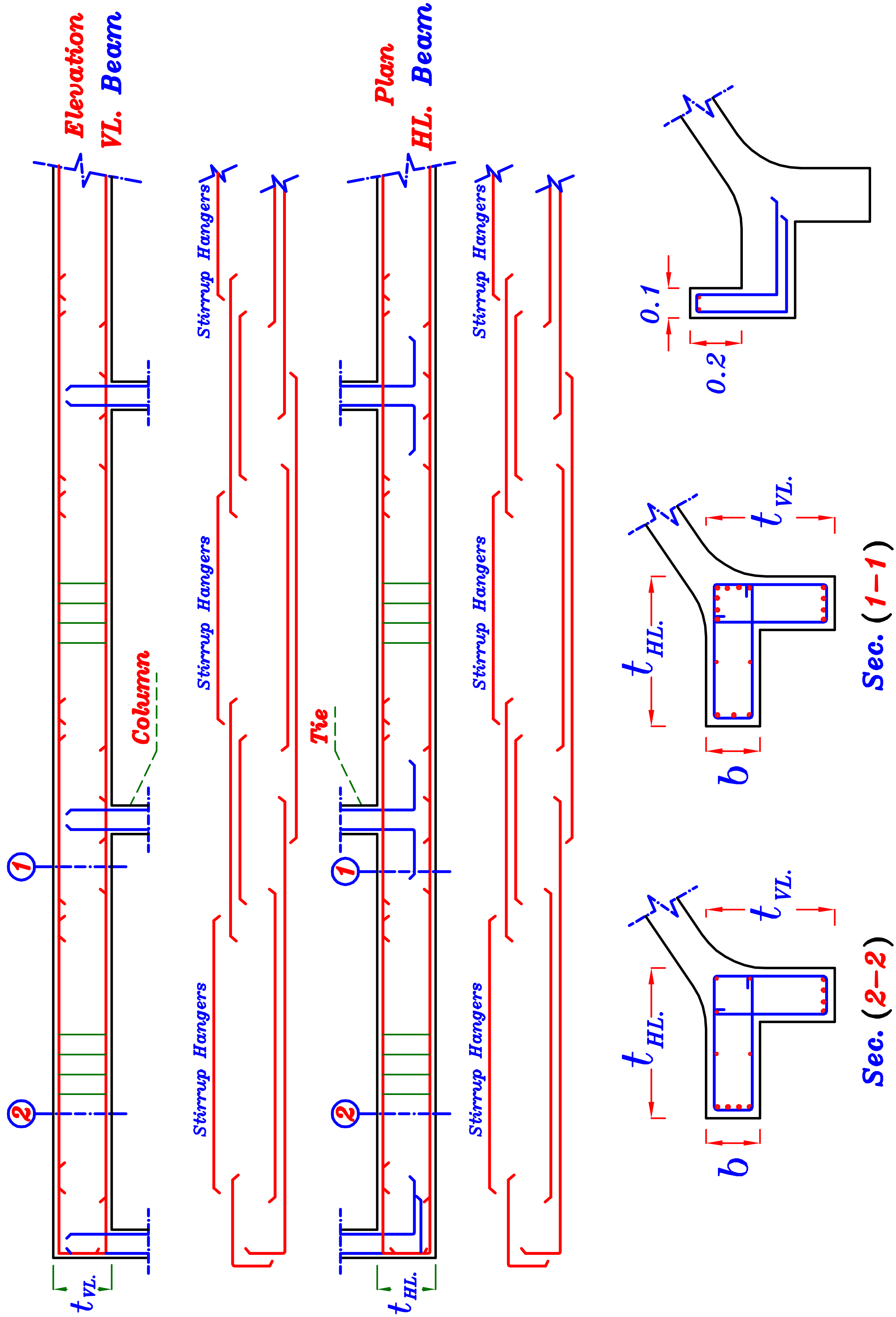




$$\boxed{R_{HL.} = X * S}$$

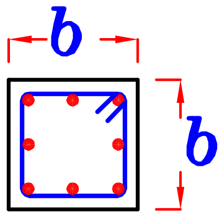
Tie تنقل الى ال

RFT. of End Beam.



* Design the Tie. $(b \times b)$

المقصود بـ b هو العرض الاصغر من عرض العمود و عرض الكمره الافقيه لان تسليح الـ tie سيدخل فى الاثنين .



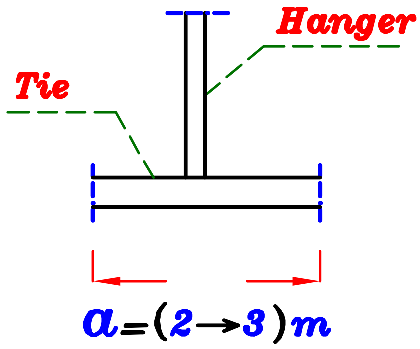
Neglect O.W. $\therefore$ B.M. $\simeq$ Zero

$$\boxed{T_{(Tie)} = R_{HL.} = X * S}$$

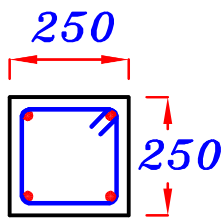
$$A_S = \frac{T_{(Tie)}}{F_y \backslash \gamma_s} = (\text{Total area of steel})$$

$$A_C = (b \times b)$$

* Design the Hanger. (250×250)



$$T = O.W._{(hanger)} + O.W._{(Tie)} * a$$

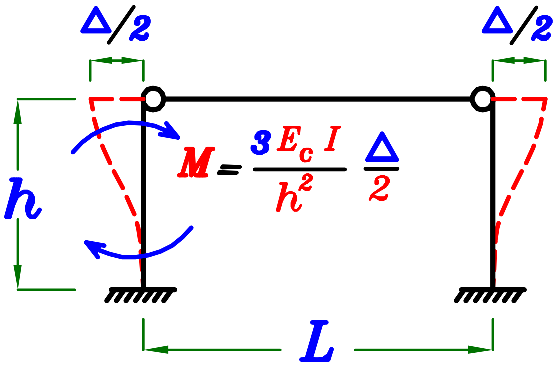


$$A_S = 4 \phi 12$$

* Design the Column.

N.F. $= R_Y$

B.M. (From the Extension of the Tie)



$$\Delta = \frac{T_{(Tie)}\, L}{E_S\, A_S} \qquad \because E_S = n\, E_c \simeq 15\, E_c$$

$$\therefore \quad \Delta = \frac{T_{(Tie)}\, L}{E_S\, A_S} = \frac{T_{(Tie)}\, L}{15\, E_c\, A_S}$$

$$\boxed{B.M. = \frac{3 E_c I}{h^2}\,\frac{\Delta}{2} = \frac{3 E_c I}{h^2}\,\frac{T_{(Tie)}\, L}{30\, E_c\, A_S} = \frac{T_{(Tie)}\, L\, I}{10\, h^2\, A_S}}$$

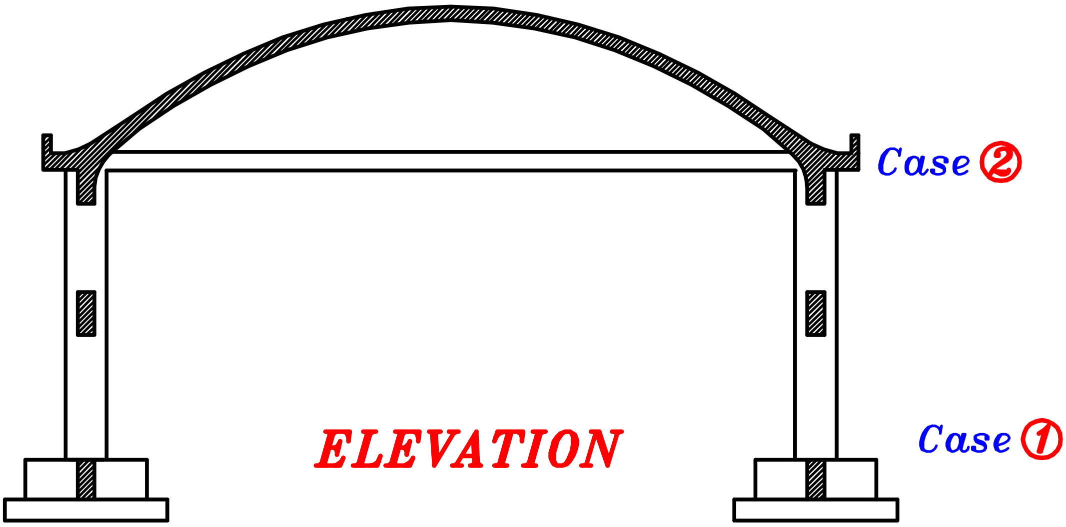
T = Tension on Tie.
L = Length of the Tie.
A_s = Area of steel of the Tie.
I = Moment of Inertia of the Column.
h = Height of the Column.

And Check Buckling. $(M_{add.})$

ممكن إهمال هذه الخطوه

We can neglect the extension of Tie.
and design the column on N.F. & M_{add} only.

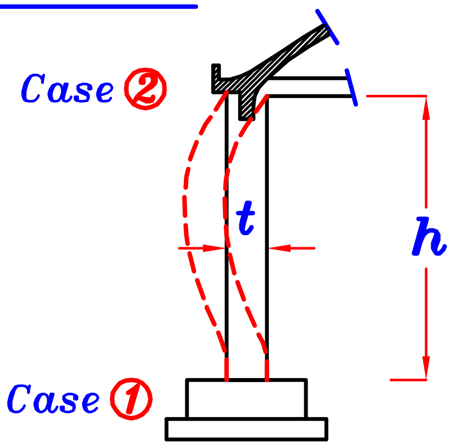
$$P = R_{VL.} = (o.w. + Y) * S$$



ELEVATION

Check Buckling.

① In Plane.

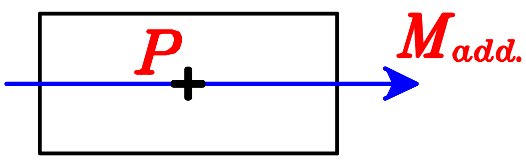


$$H_o = h$$

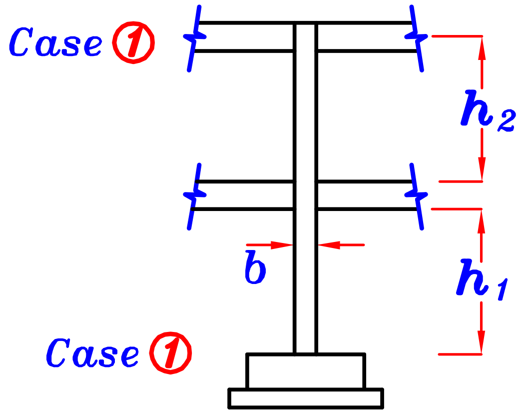
$$\lambda_b = \frac{1.3 * H_o}{t}$$

IF $\lambda_b \leqslant 10 \xrightarrow{\text{Designed}} P$ only

$\lambda_b > 10 \xrightarrow{\text{Designed}} P , M_{add.}$



② Out of Plane.

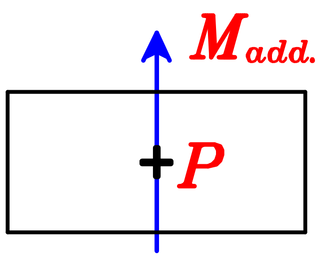


$H_o =$ The bigger of h_1 , h_2

$$\lambda_b = \frac{1.2 * H_o}{b}$$

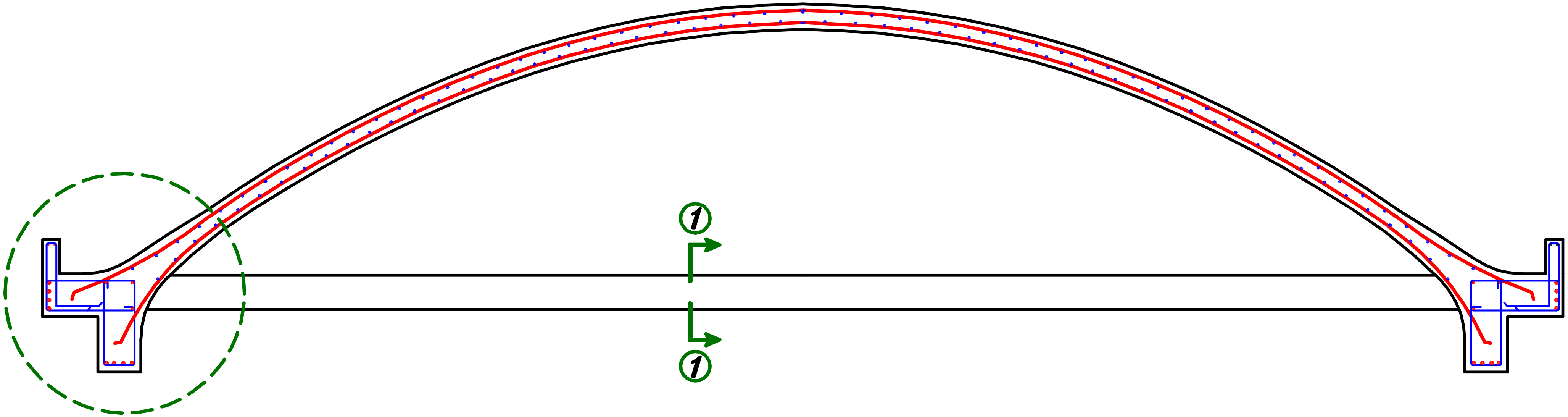
IF $\lambda_b \leqslant 10 \xrightarrow{\text{Designed}} P$ only

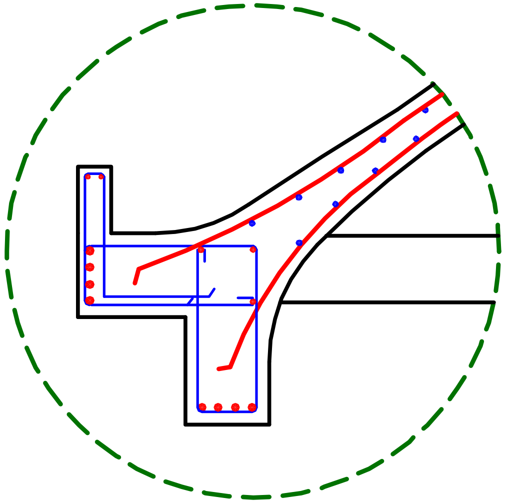
$\lambda_b > 10 \xrightarrow{\text{Designed}} P , M_{add.}$



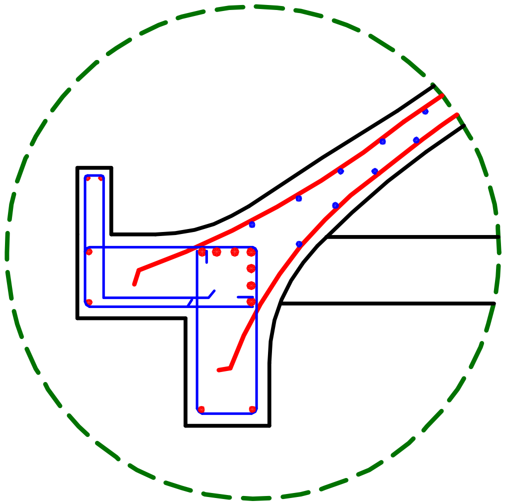
Reinforcement of Arch Slab.

Without hangers or Stiffener

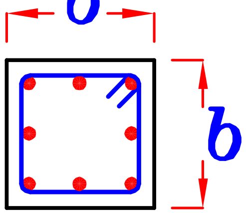




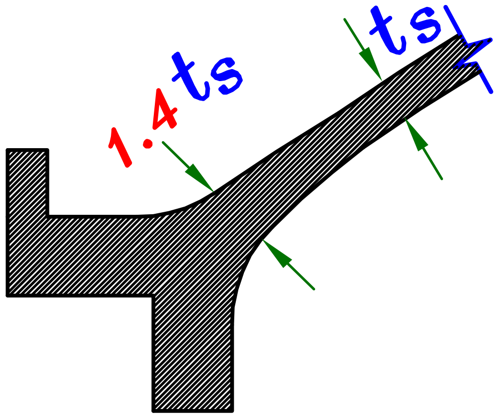
Sec. at
the Mid Span.



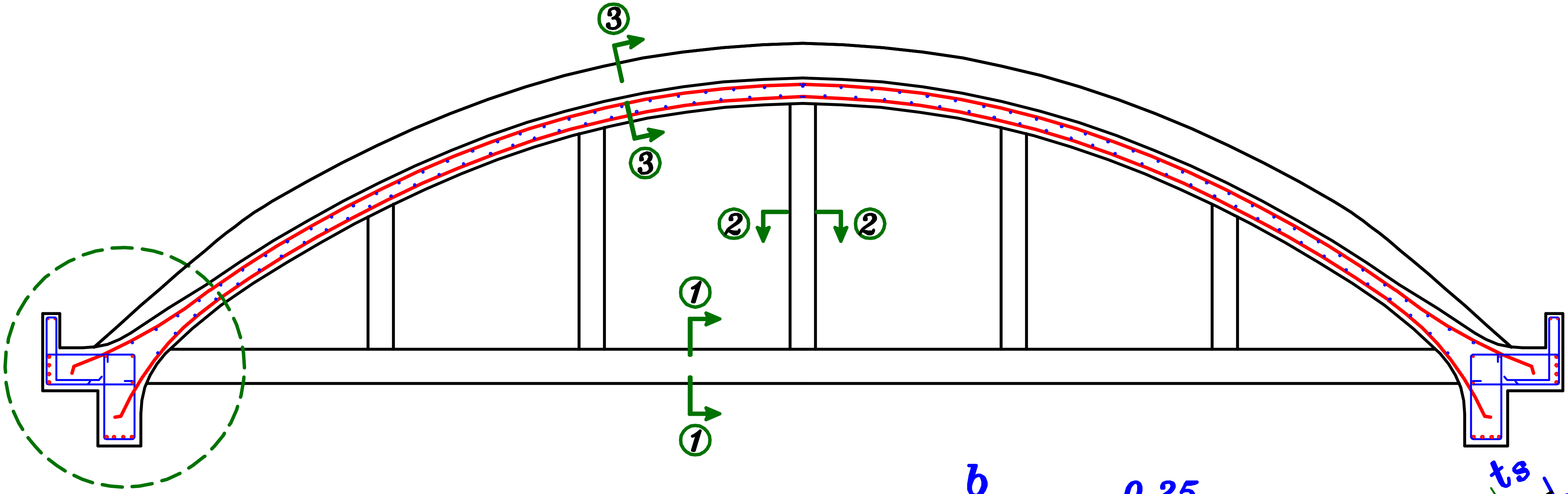
Sec. at
the Column.

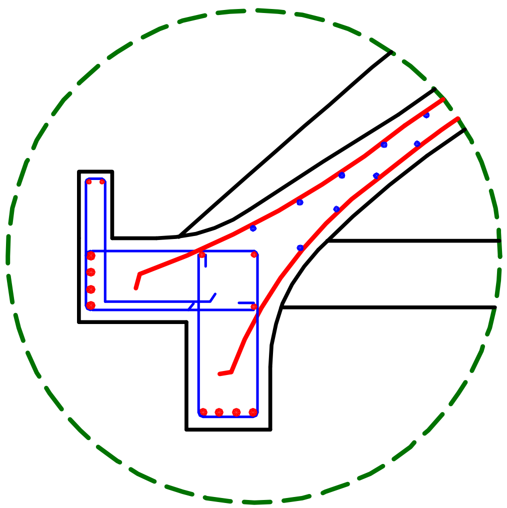


Sec. (1−1)

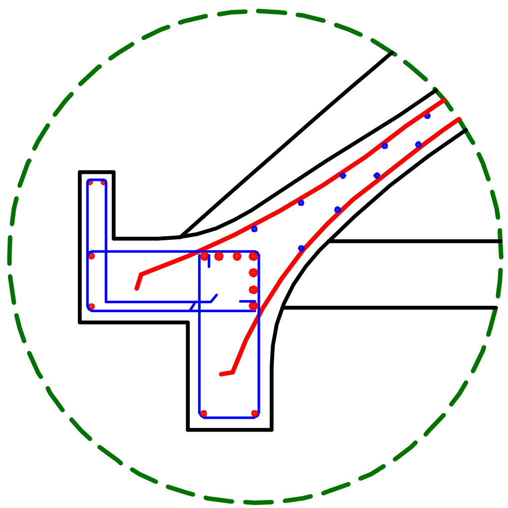


Reinforcement of Arch Slab. With hangers and Stiffener

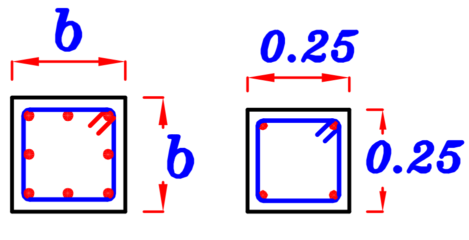




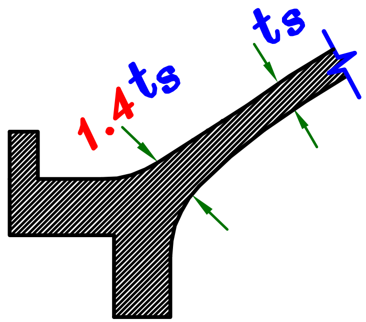
Sec. at the Mid Span.

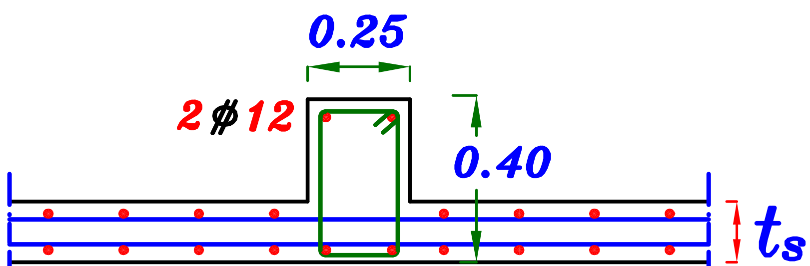


Sec. at the Column.



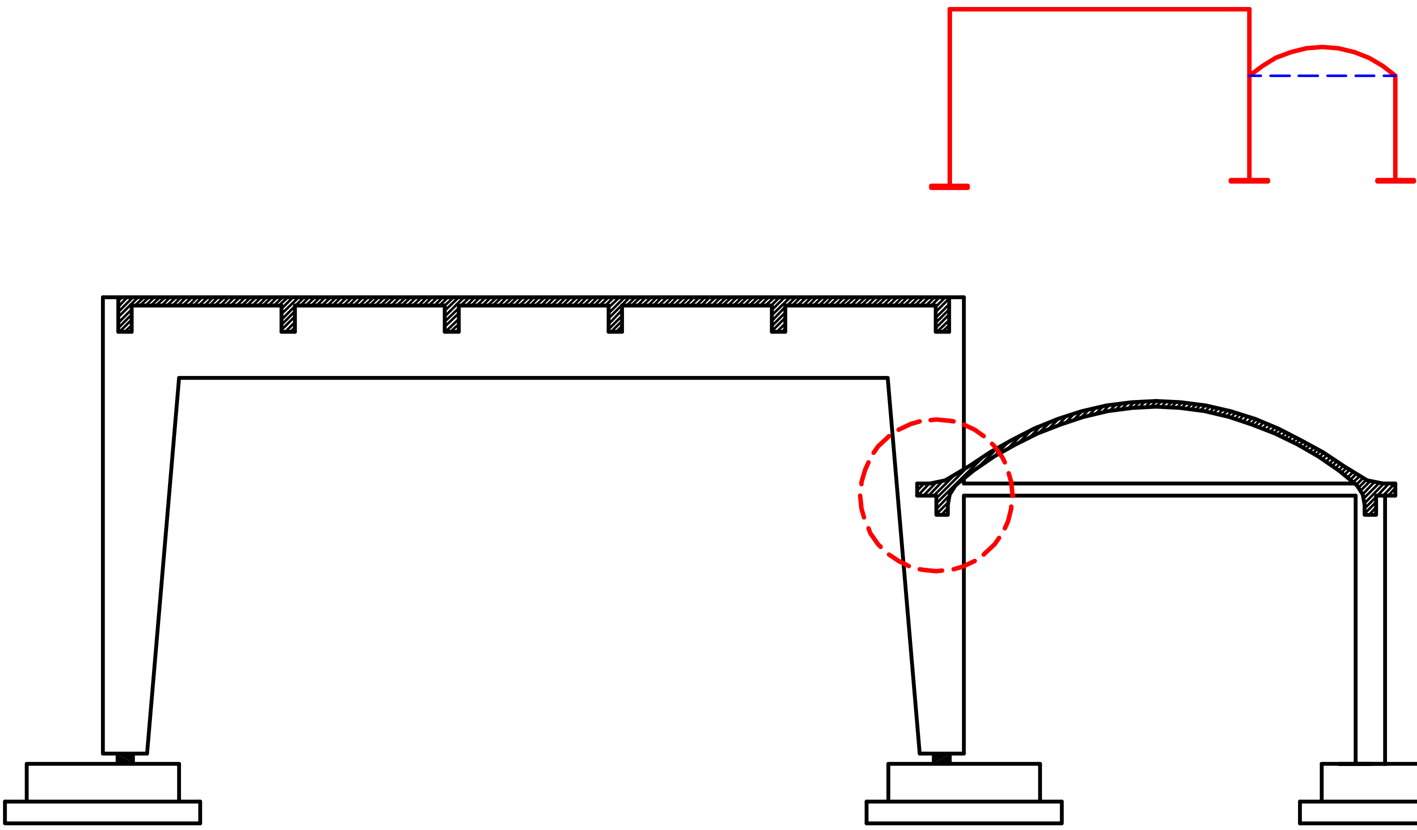
Sec. (1–1) Sec. (2–2)

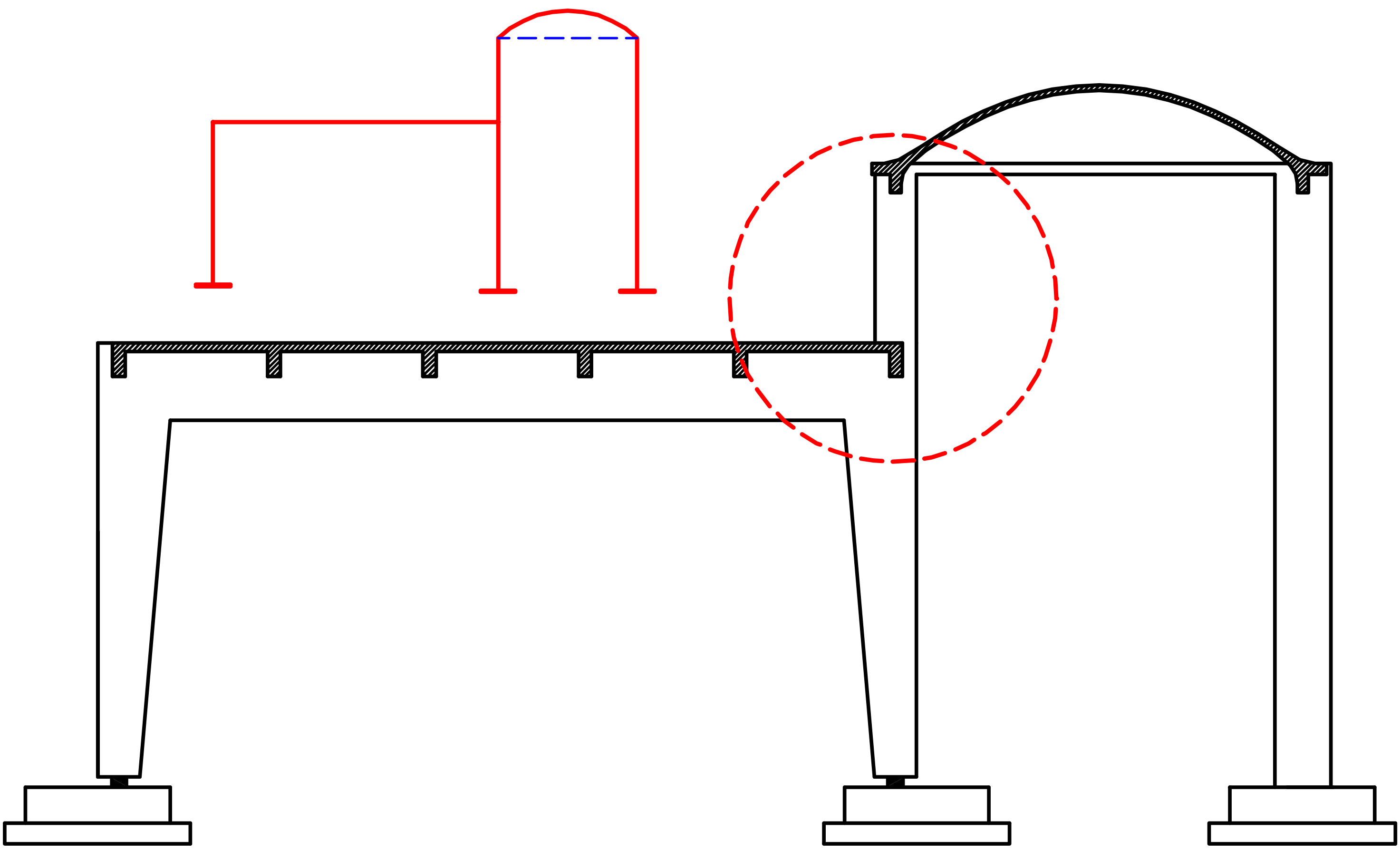




Sec. (3–3)

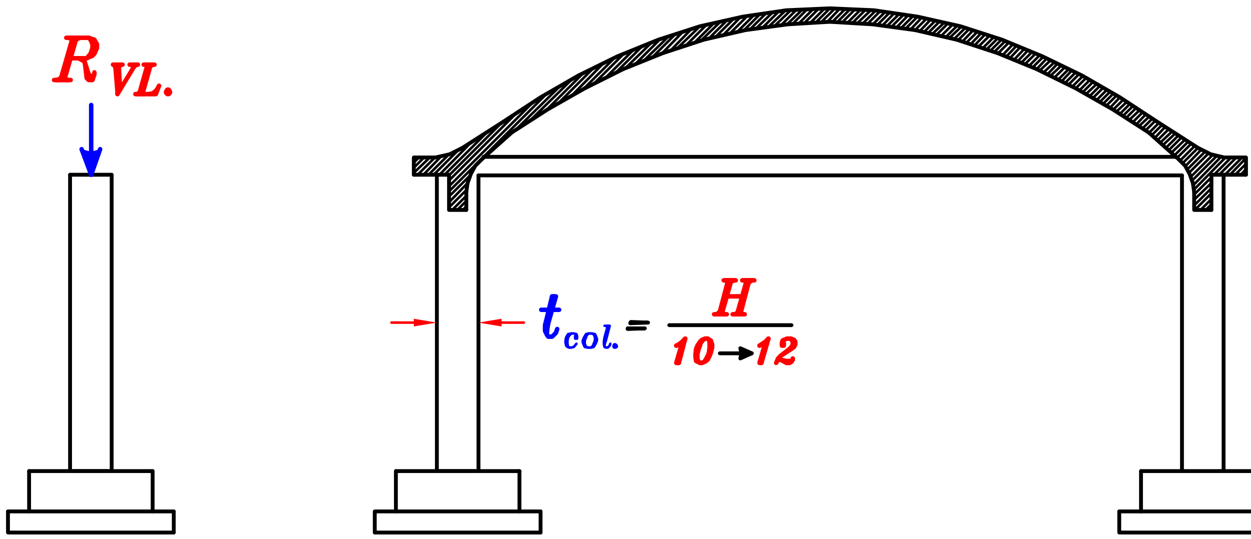
Connection between Arch slab & Frame.





Special Cases.

1- Arch Slab without Tie.



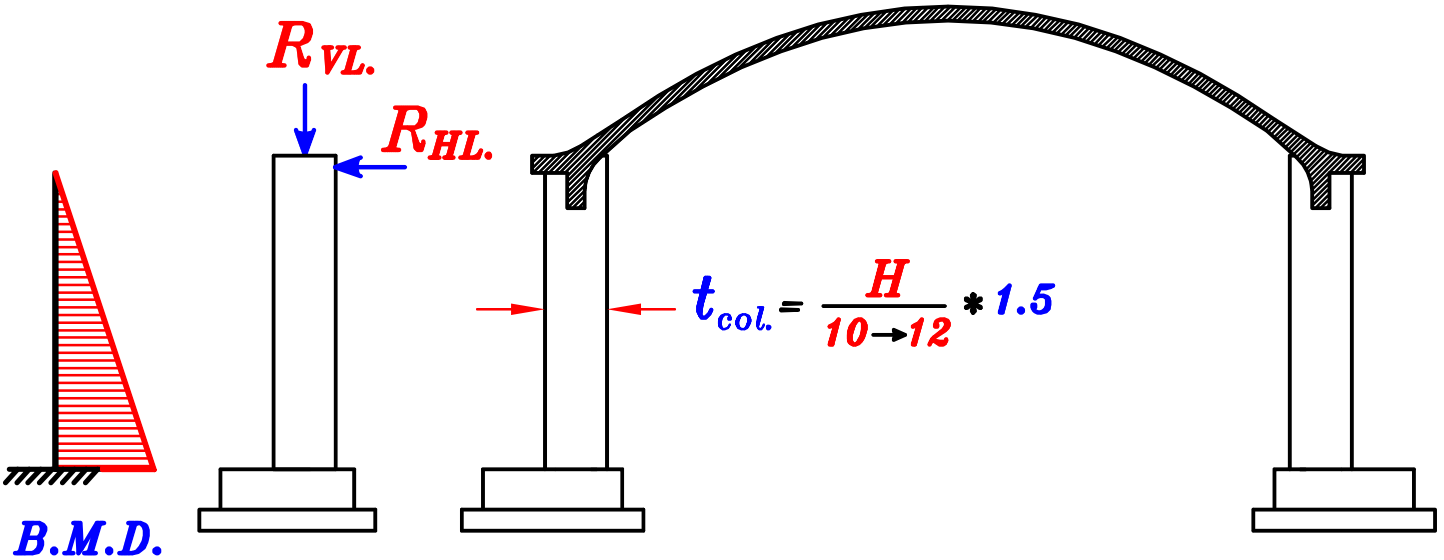
اذا لم نضع **Tie** مع ال **Arch Slab** سينتقل الحمل من الكمره الرأسيه $R_{VL.}$

الى العمود ليعمل **Normal Force** على العمود .

و ستنتقل القوى الافقيه من الكمره الافقيه $R_{HL.}$ الى العمود أيضا لتعمل

Bending Moment على العمود .

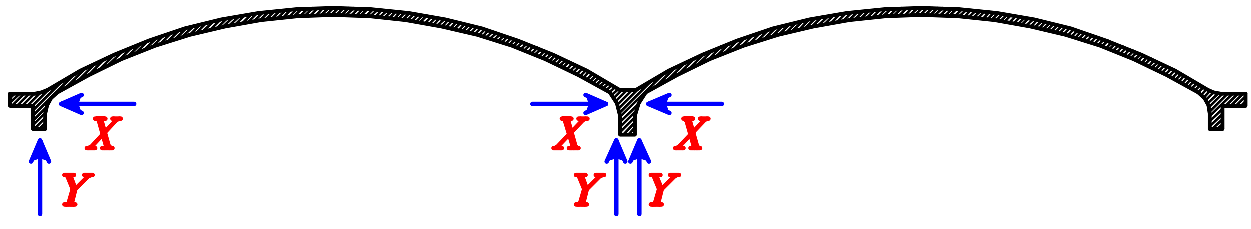
فيتم تصميم العمود على M, N و يتم ترحيل القاعده عكس ال **moment** .

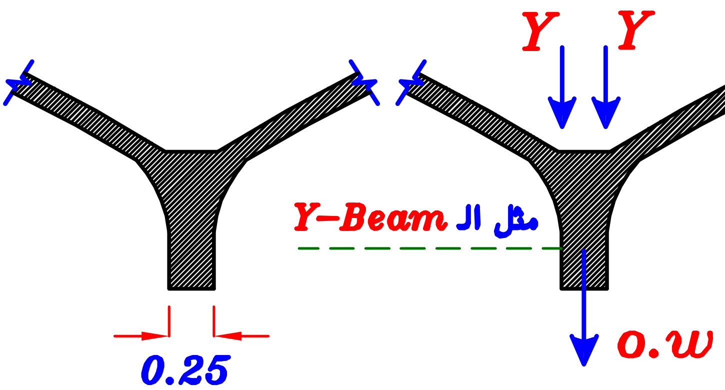


2-Continuous Arch Slab.

اذا وجدت بلاطتان **Arch Slab** متجاورتان و متساويتان فى الابعاد
تكون الكمره بينهم كمره **Vertical** و لا توجد كمره **Horizontal** لانه لا توجد **X** على الكمره
و تكون هذه الكمره مثل ال **Y-Beam** و نأخذ تخانتها

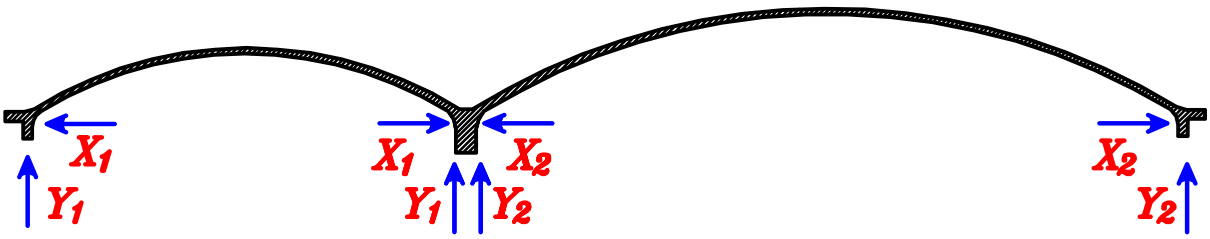
$$\boxed{t_{Y-beam} \simeq \frac{Spacing}{12} + 150\ mm}$$

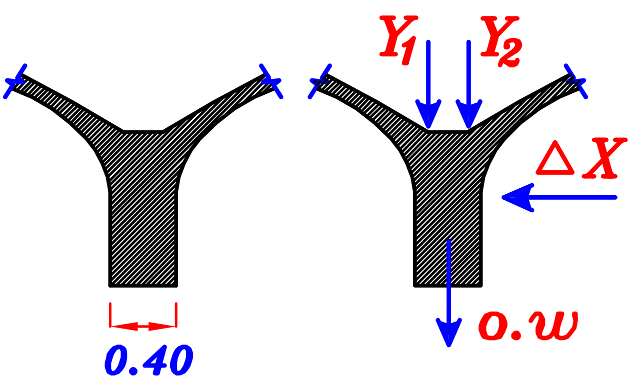




$$\boxed{w_{VL.} = o.w. + 2Y}$$

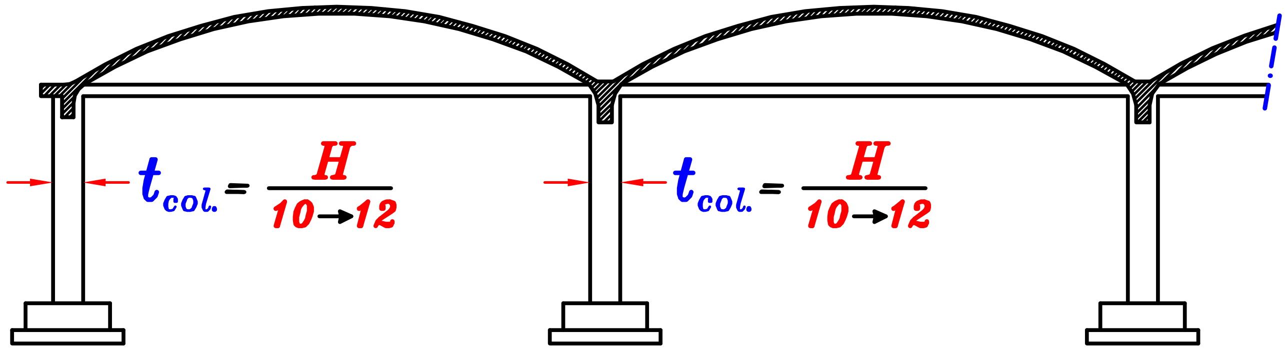
اذا وجدت بلاطتان **Arch Slab** متجاورتان و لكن غير متساويتان فى الابعاد
تكون الكمره بينهم كمره **Vertical** و لا توجد كمره **Horizontal**
و لكن نجعل عرض الكمره ال **Vertical** تساوى على الاقل ٤٠ سم حتى تتحمل فرق القوى الافقيه
و تصمم الكمره على **Bi-Axial moment**

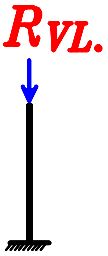




$$\boxed{\begin{aligned} w_{VL.} &= o.w. + Y_1 + Y_2 \\ w_{HL.} &= \triangle X = X_2 - X_1 \end{aligned}}$$

3_ Continuous Arch Slab with Tie.

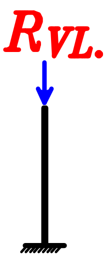




$$\boxed{R_{VL.} = (o.w. + 2\,Y) * S}$$

الاعمده الداخليه لا يوجد عليها moment

لا يوجد ترحيل للقواعد

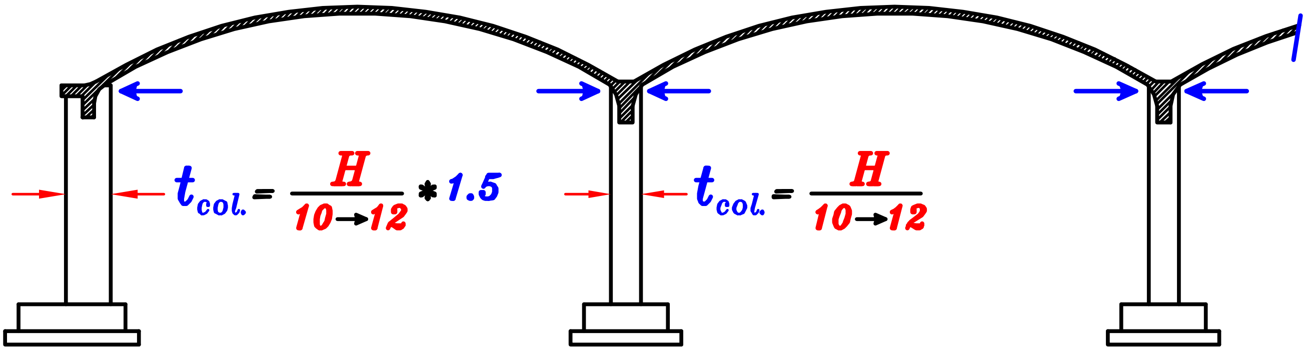


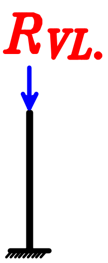
$$\boxed{R_{VL.} = (o.w. + Y) * S}$$

الاعمده الخارجيه لا يوجد عليها moment

لا يوجد ترحيل للقواعد

4_ Continuous Arch Slab without Tie.

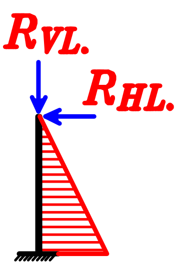




$$\boxed{R_{VL.} = (o.w. + 2\,Y) * S}$$

الاعمده الداخليه لا يوجد عليها moment

لا يوجد ترحيل للقواعد



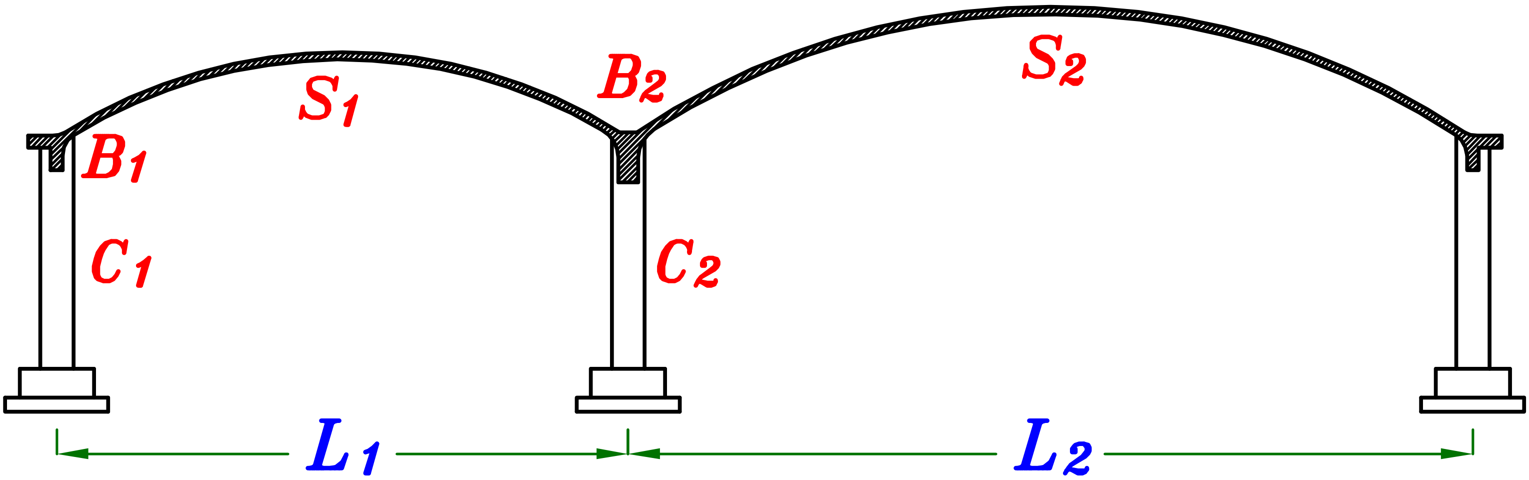
$$\boxed{R_{VL.} = (o.w. + Y) * S}$$

$$\boxed{R_{HL.} = X * S}$$

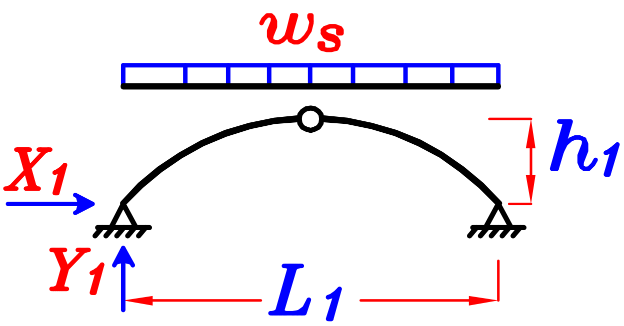
الاعمده الخارجيه يوجد عليها moment

ترحل القواعد للخارج عكس ال moment

IF the Arch Slabs are not equal.



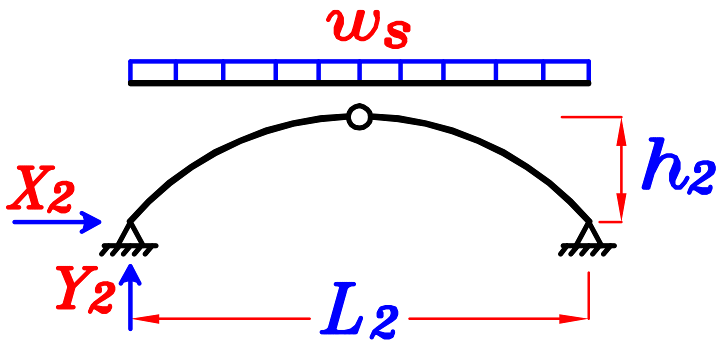
Arch Slab S_1



$$Y_1 = \frac{w_s\, L_1}{2}$$

$$X_1 = \frac{w_s\, L_1^2}{8\, h_1}$$

Arch Slab S_2



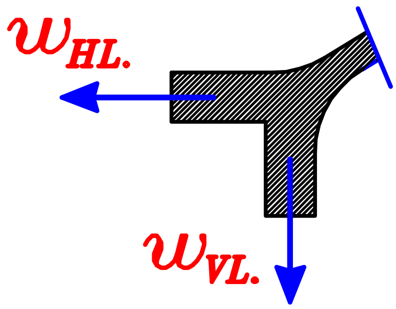
$$Y_2 = \frac{w_s\, L_2}{2}$$

$$X_2 = \frac{w_s\, L_2^2}{8\, h_2}$$

B_1

$$w_{VL.} = o.w. + Y_1$$

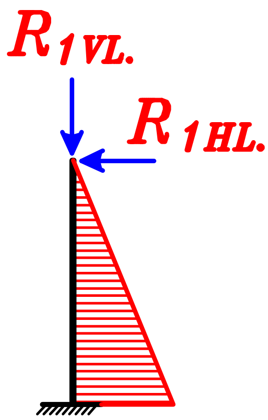
$$w_{HL.} = X_1$$



C_1

$$R_{1\,VL.} = (o.w. + Y_1) * S$$

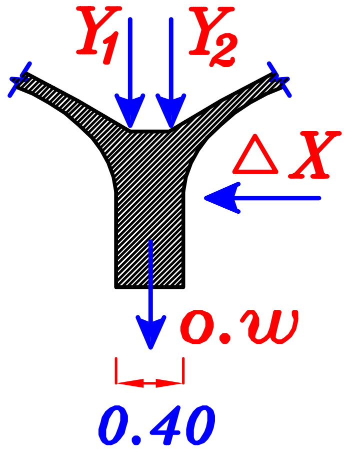
$$R_{1\,HL.} = X_1 * S$$



B_2

$$w_{VL.} = o.w. + Y_1 + Y_2$$

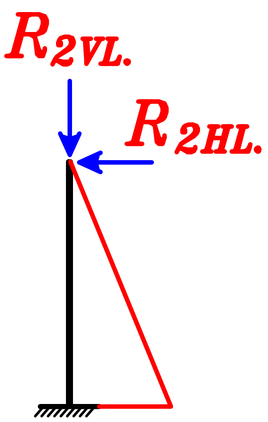
$$w_{HL.} = \triangle X = X_2 - X_1$$



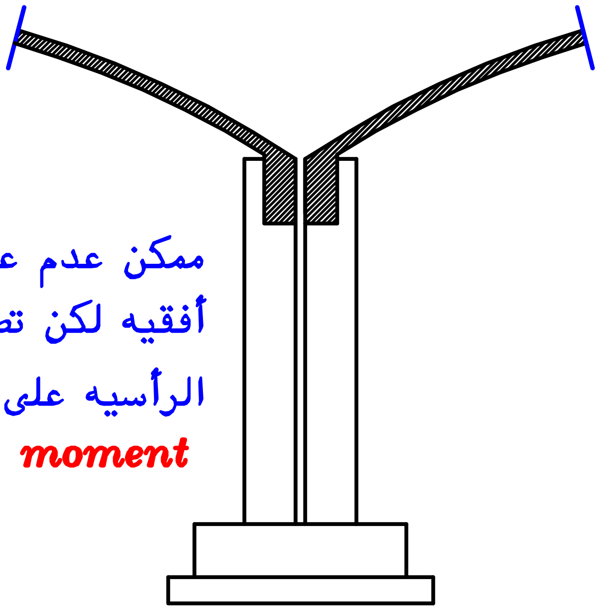
C_2

$$R_{2\,VL.} = (o.w. + Y_1 + Y_2) * S$$

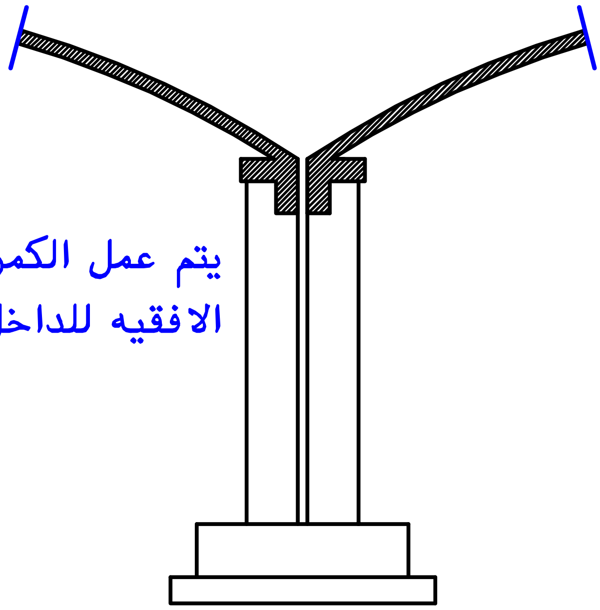
$$R_{2\,HL.} = \triangle X * S$$



5- Expansion Joint in continuous Arch Slab.

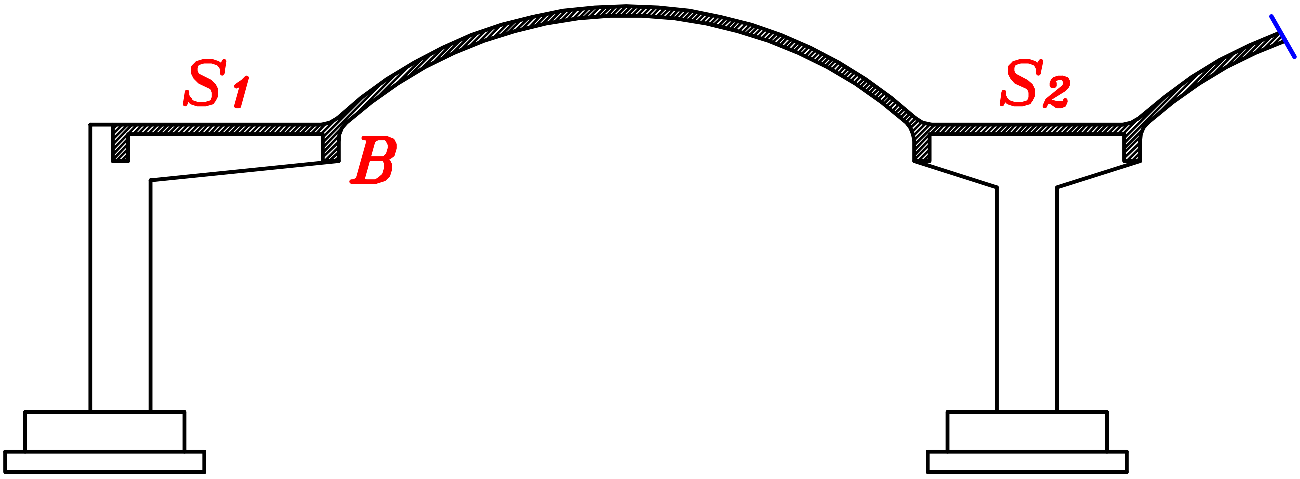


ممكن عدم عمل كمرات
أفقيه لكن تصمم الكمره
الرأسيه على
Bi-Axial moment

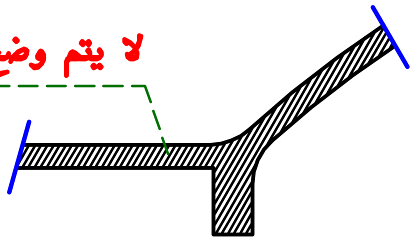


يتم عمل الكمرات
الافقيه للداخل

6-HL. Slab connected to Arch Slab.

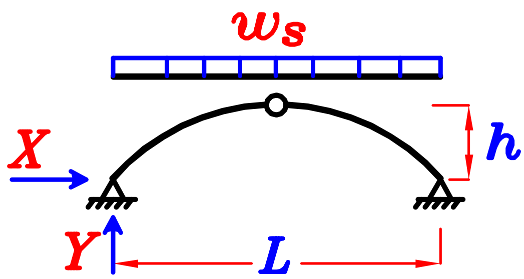


<u>B</u>



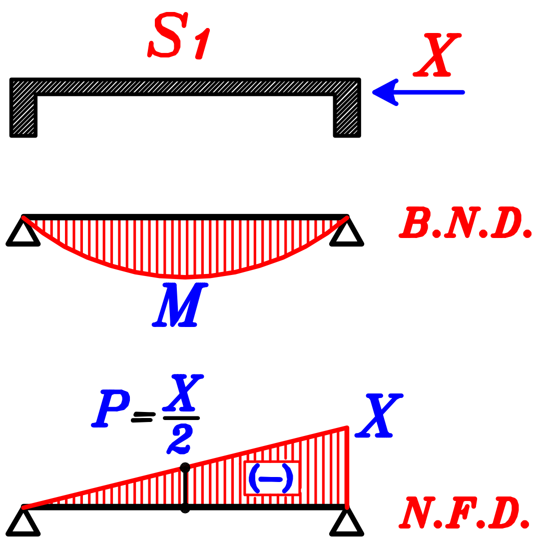
لا يتم وضع كمره أفقيه لان البلاطه الافقيه تعمل عمل الكمره الافقيه .

HL. Slabs.

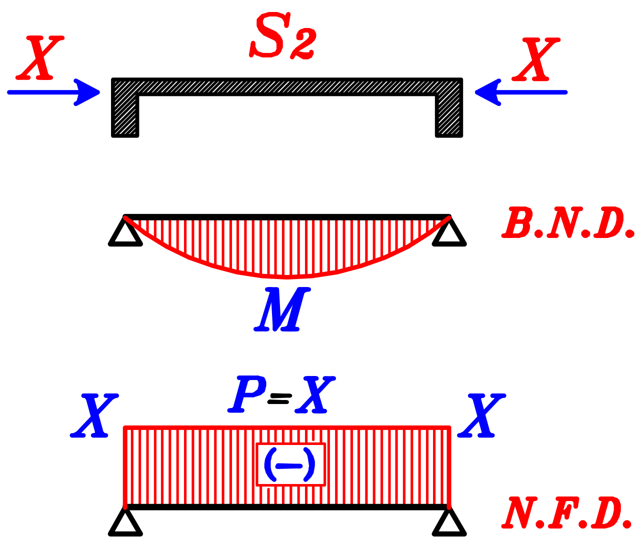


يفضل زياده تخانتها حوالى ٥٠ مم لمقاومه ال **buckling**
و يتم تصميمها بال *I.D.* و يكون تسليحها شبكتين متساويتين

Strip 1.0 m

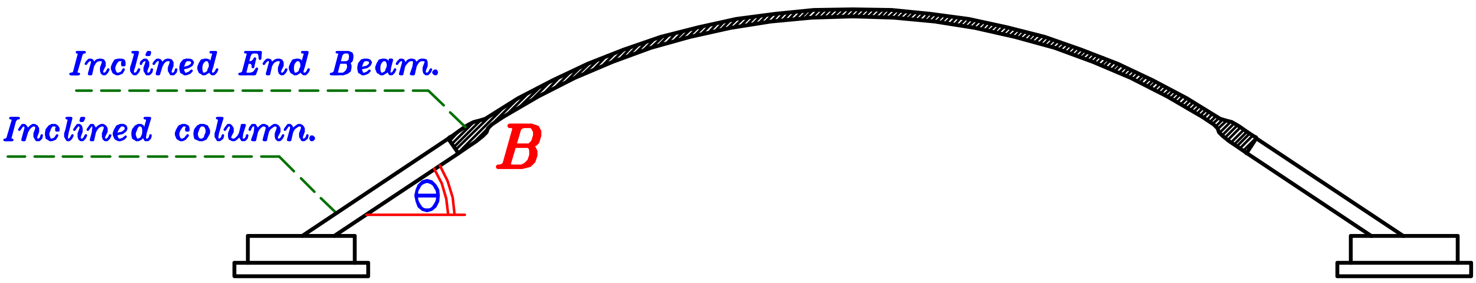


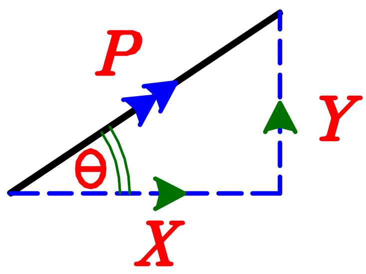
Design the slab on *M*, *P*
using *I.D.*

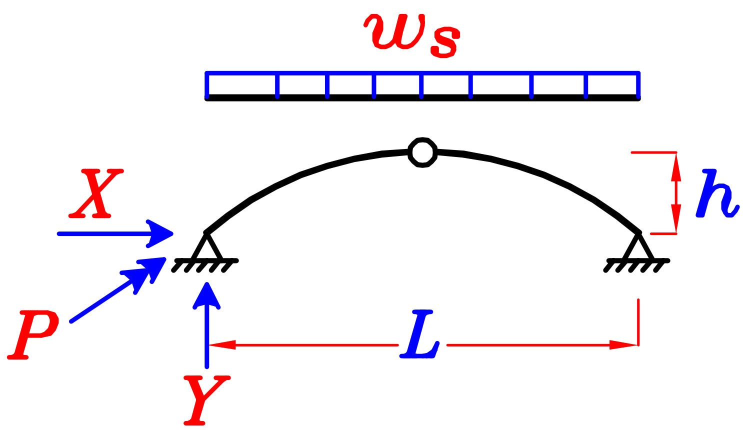


Design the slab on *M*, *P*
using *I.D.*

7- Inclined End Beam.





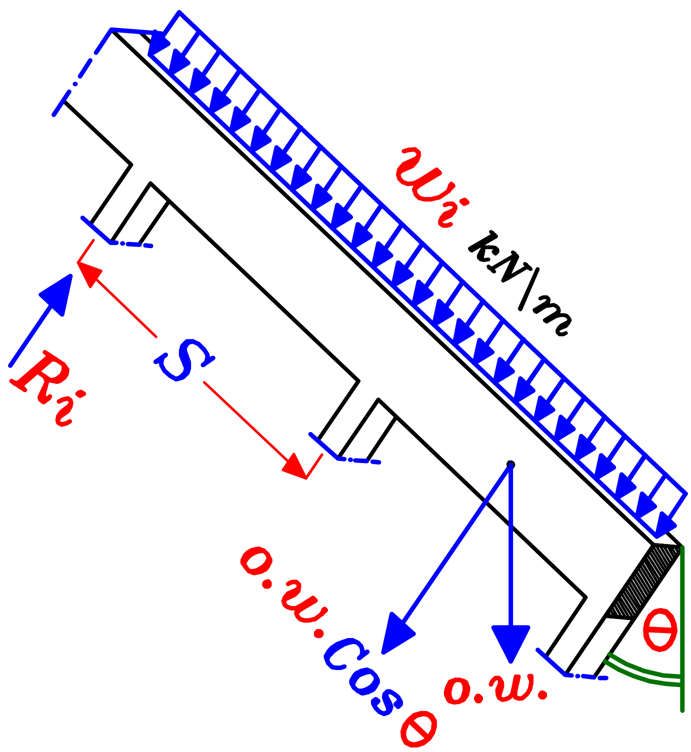


$$P = \sqrt{X^2 + Y^2}$$

Inclined Beam B

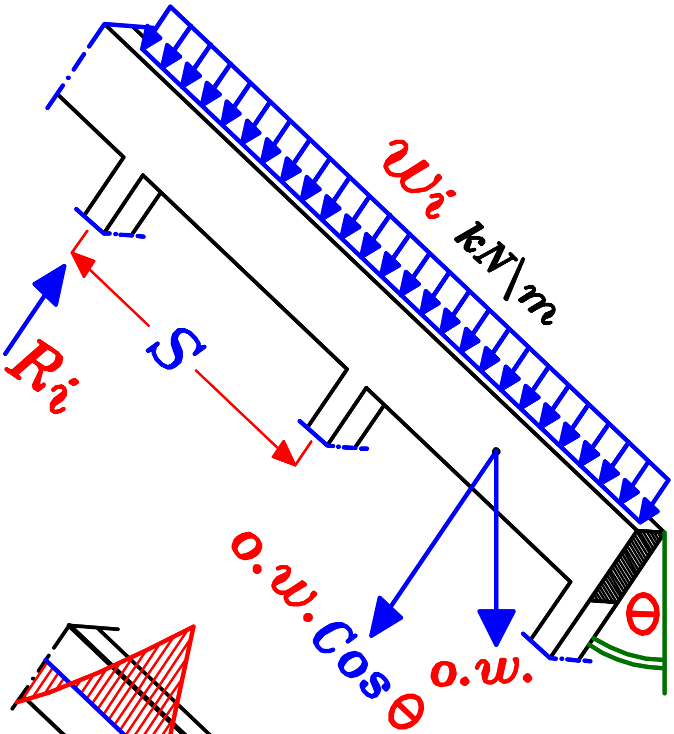
$$w_i = P + o.w. * \cos\theta$$

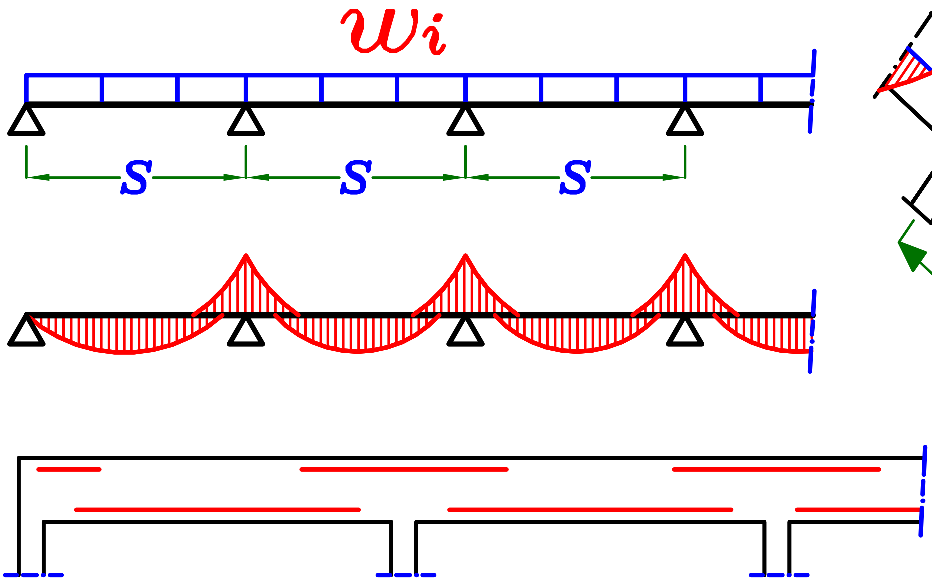
$$R_i = w_i * S$$

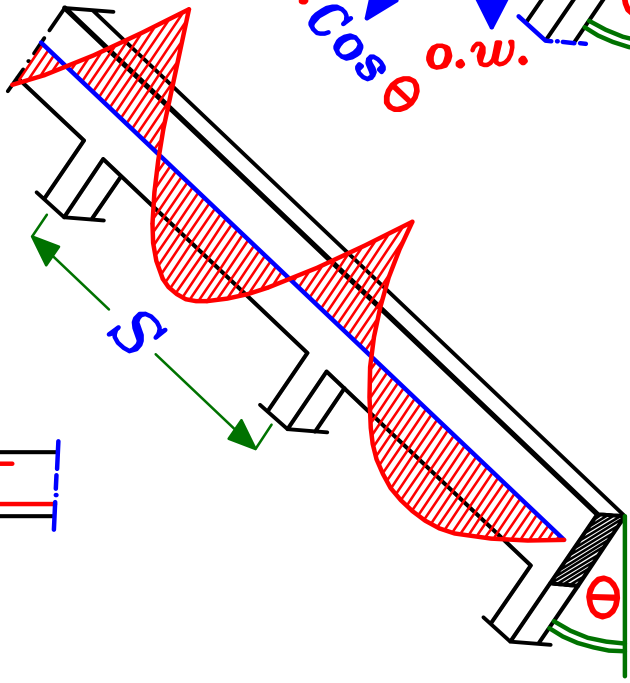


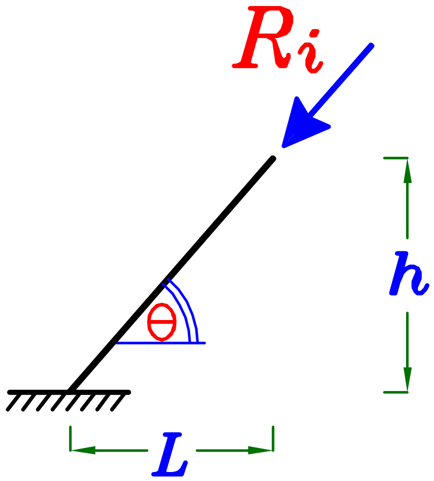
$$\boxed{w_i = P + o.w. * \cos\theta}$$

$$\boxed{R_i = w_i * S}$$







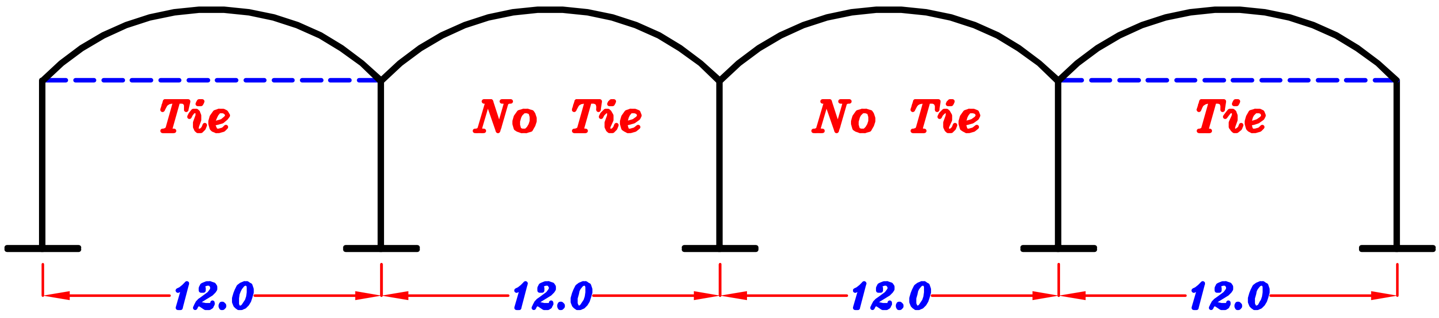


اذا كان ميل العمود هو نفس ميل المحصله
لن يكون هناك moment على العمود .

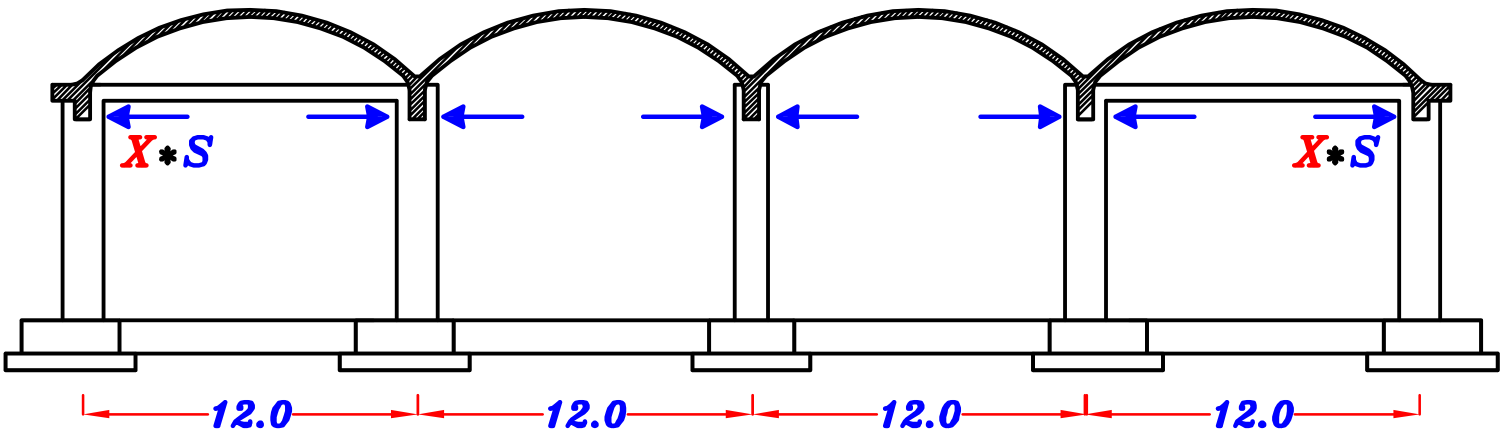
اذا كان ميل العمود ليس نفس ميل المحصله
سيكون هناك moment على العمود .

> لن يتم وضع Tie حتى لا تسحب X
> حتى تكون المحصله نفس ميل العمود

Example.



Choose a convenient Statical System and draw a sketch For an elevation Showing Concrete Dimensions.

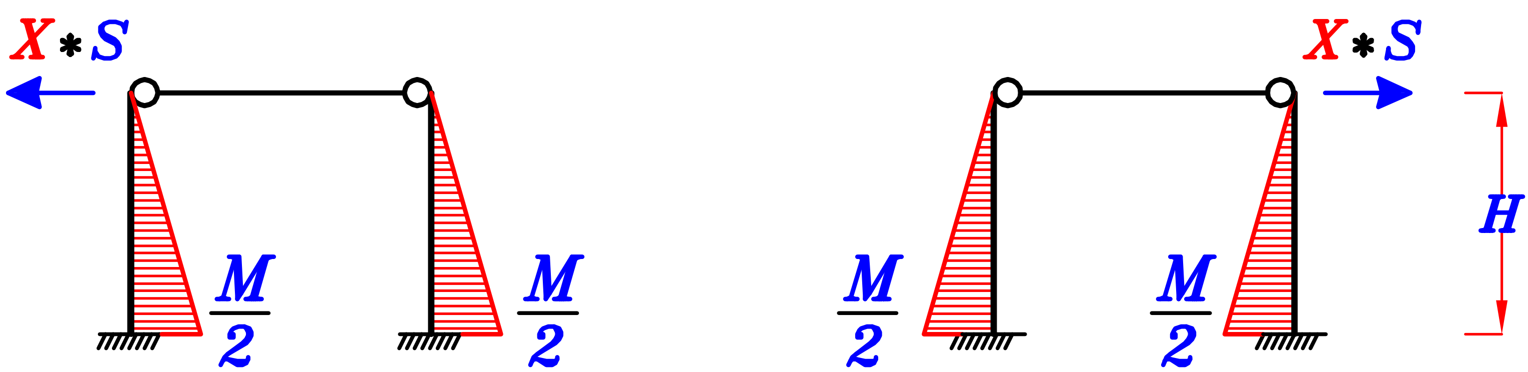


اذا تم ازاله ال Tie فى الباكيتين اللتان فى المنتصف فقط.

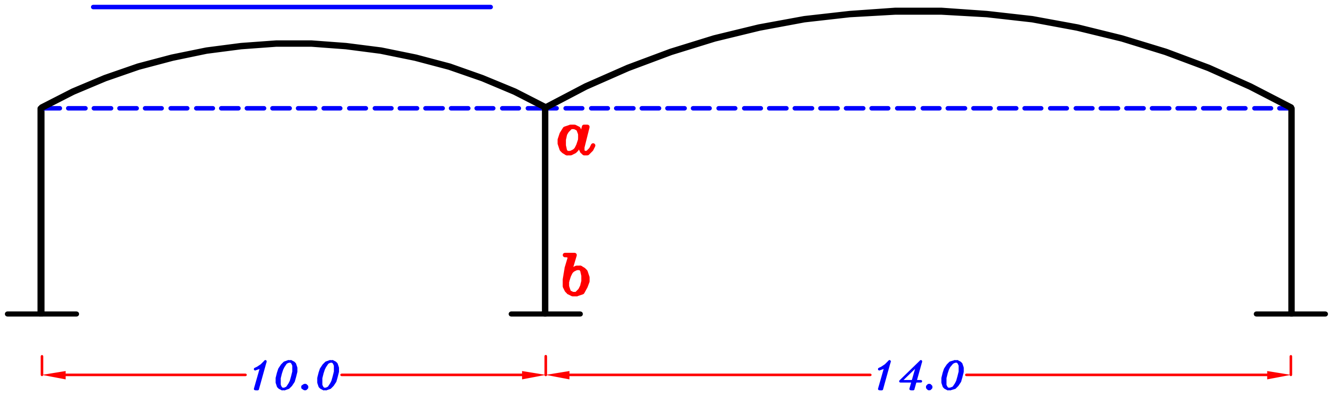
ستكون كل Tie فى الاطراف غير متزنه داخليا فى اتجاه X

لذلك سيتكون عزم تتوزع على الاعمده بالتساوى .

$$\boxed{M = (X * S) * H}$$



Example.



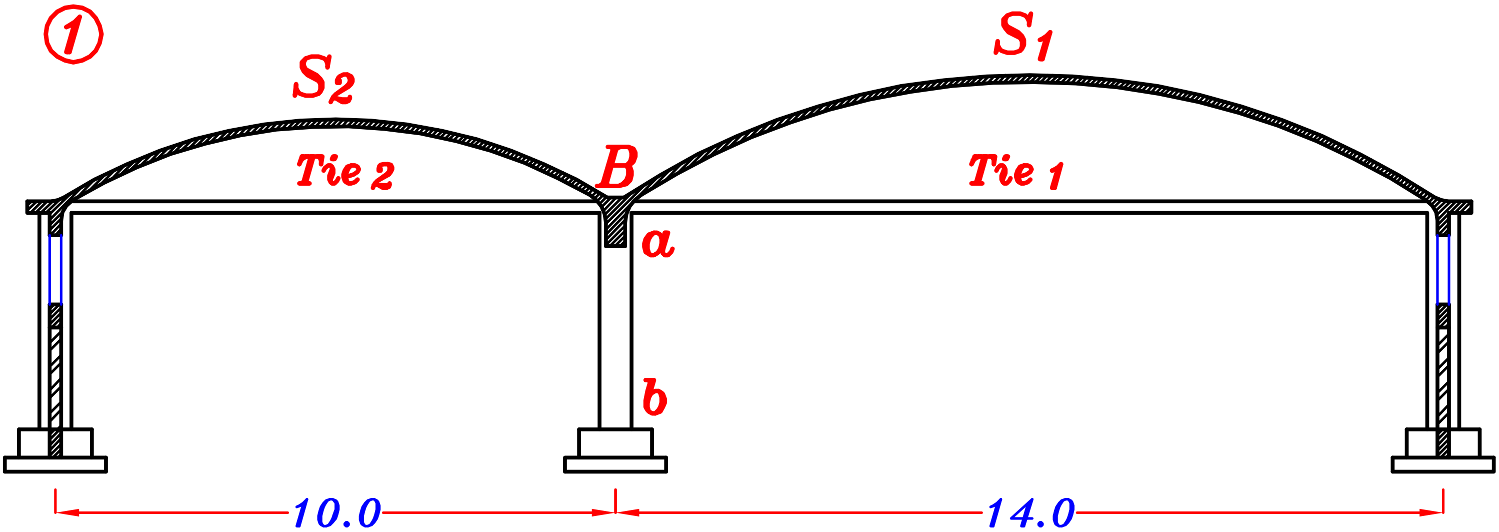
Required.

1_ Draw concrete dimensions For the given system.

2_ Remove the ties then draw concrete dimensions.

3_ Remove column a b then choose a convenient system.

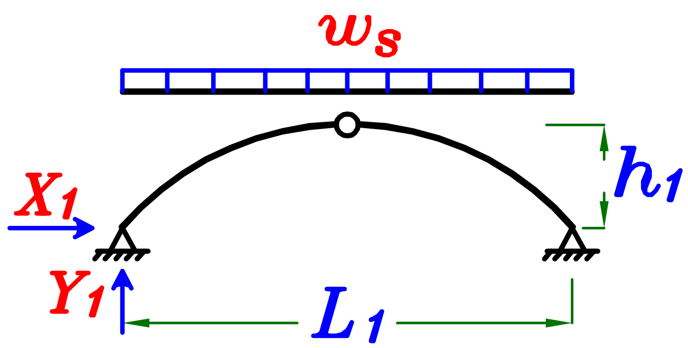
(1)



Arch Slab S_1

$$Y_1 = \frac{w_s L_1}{2}$$

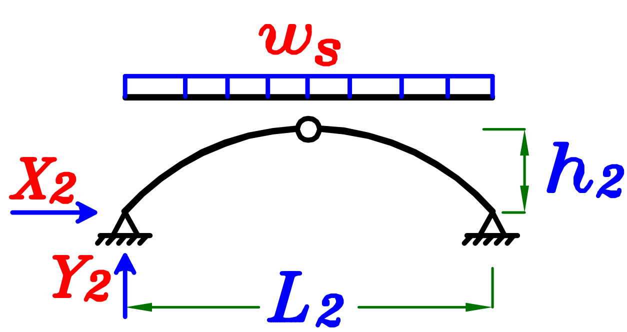
$$X_1 = \frac{w_s L_1^2}{8 h_1}$$



Arch Slab S_2

$$Y_2 = \frac{w_s L_2}{2}$$

$$X_2 = \frac{w_s L_2^2}{8 h_2}$$

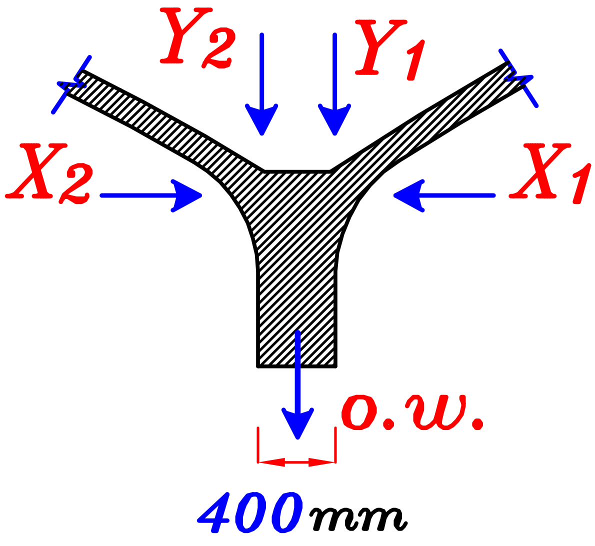


Beam B

Take $b = 400\ mm$

$w_{VL.} = o.w. + Y_1 + Y_2$

$w_{HL.} = \triangle X = X_1 - X_2$



Reactions of beam B

$R_{VL.} = w_{VL.} * S$

$R_{HL.} = w_{HL.} * S$

Tie 1

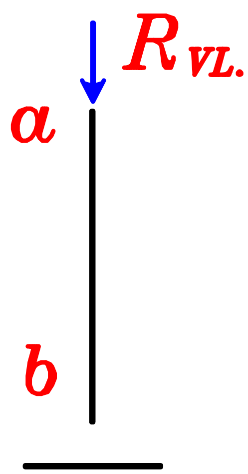
$T_1 = X_1 * S$

Tie 2

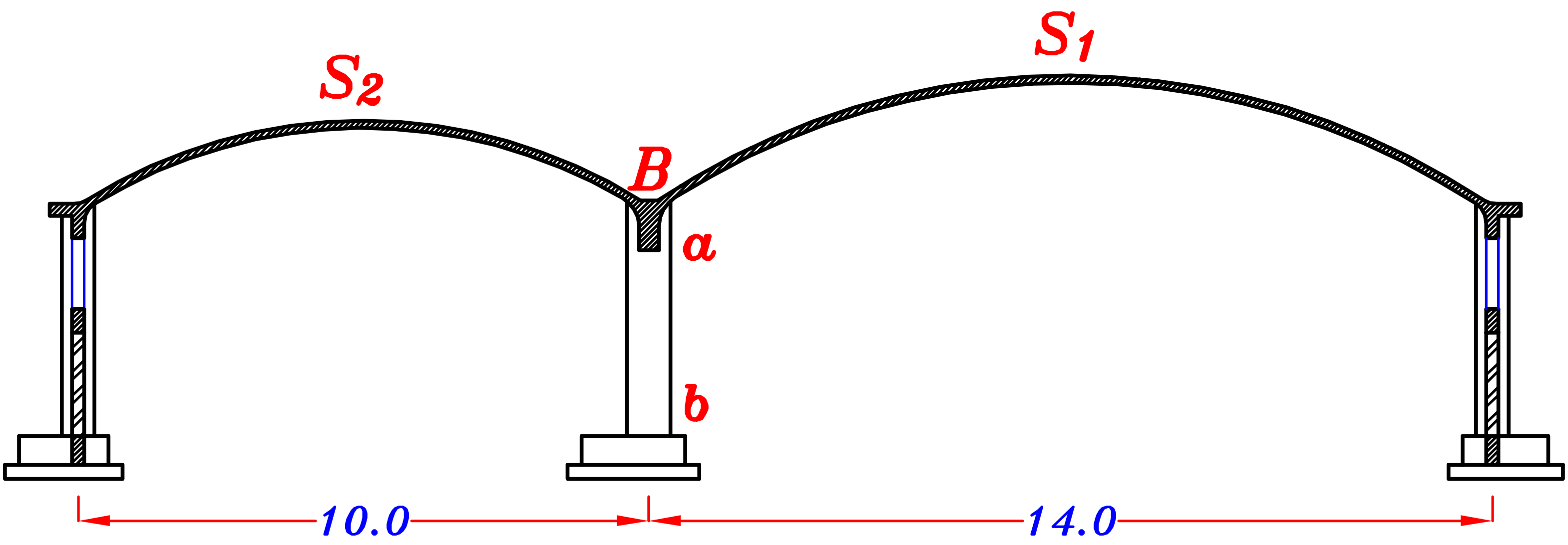
$T_2 = X_2 * S$

Column ab

Normal $= R_{VL.} = w_{VL.} * S$



2- Remove the ties.

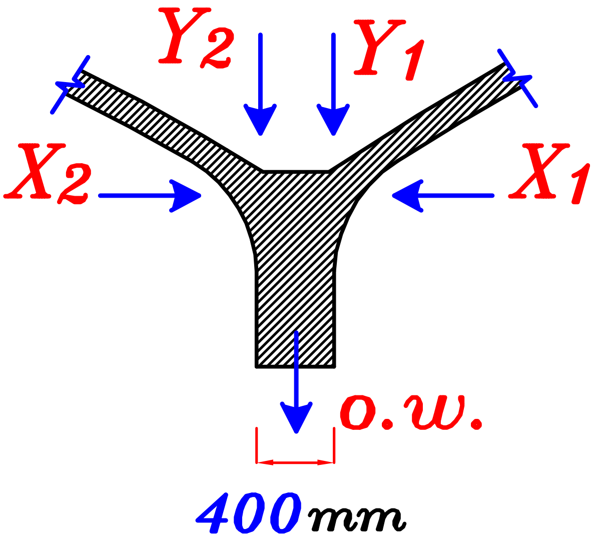


Beam B

Take $b = 400$ mm

$w_{VL.} = o.w. + Y_1 + Y_2$

$w_{HL.} = \triangle X = X_1 - X_2$

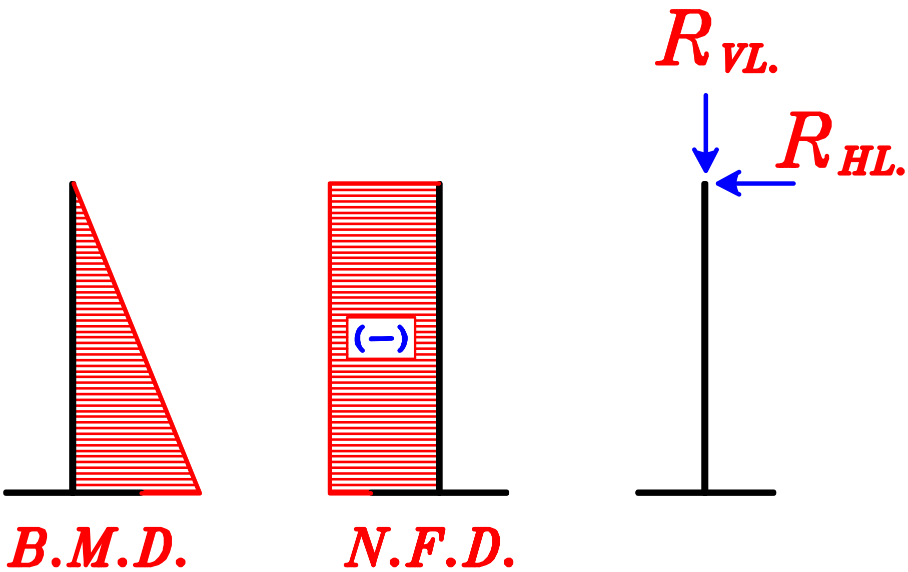


Reactions of beam B

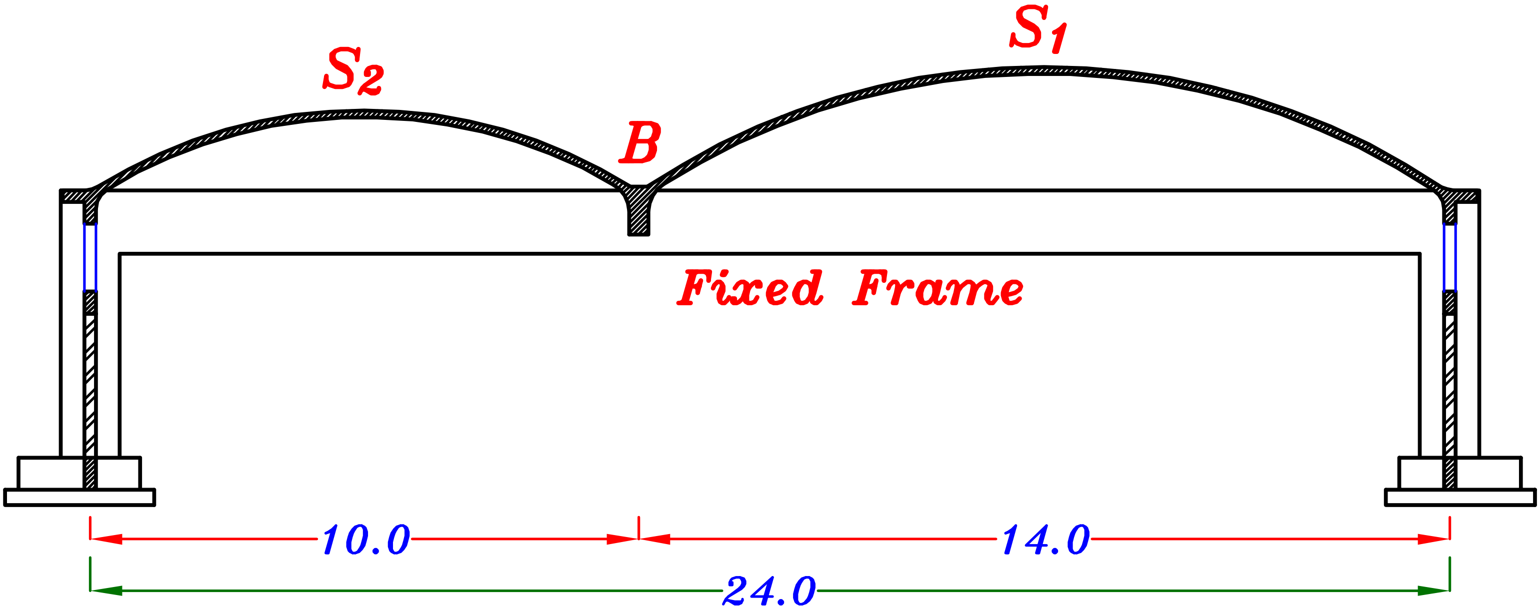
$R_{VL.} = w_{VL.} * S$

$R_{HL.} = w_{HL.} * S = \triangle X * S$

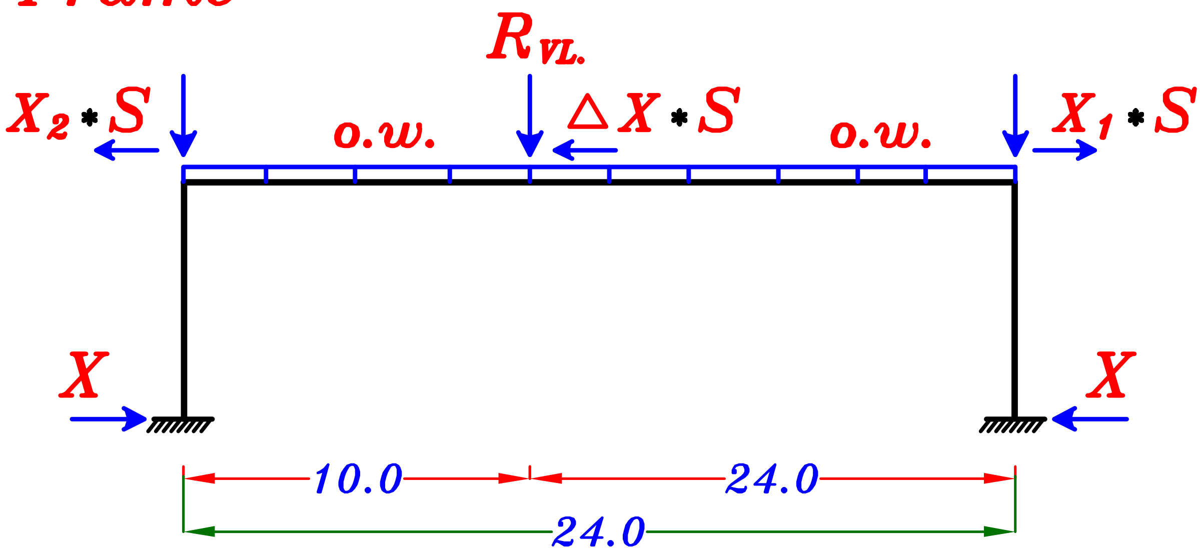
Column a b

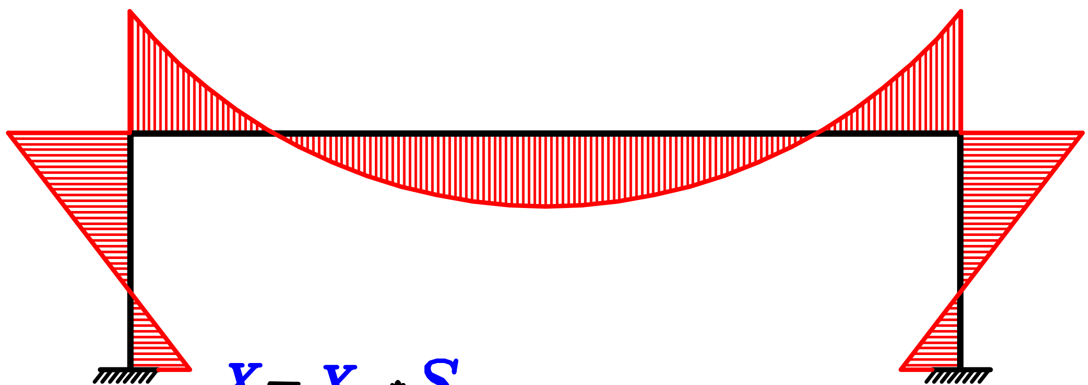


3- Remove the column a b

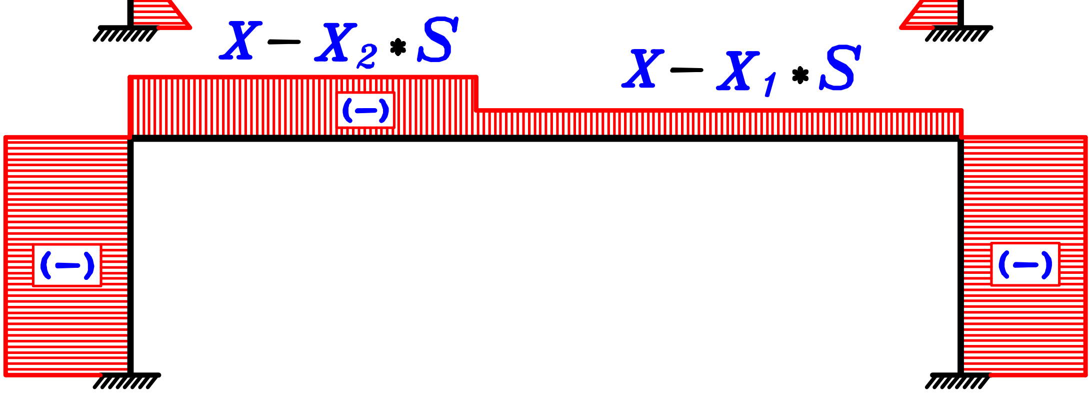


Fixed Frame





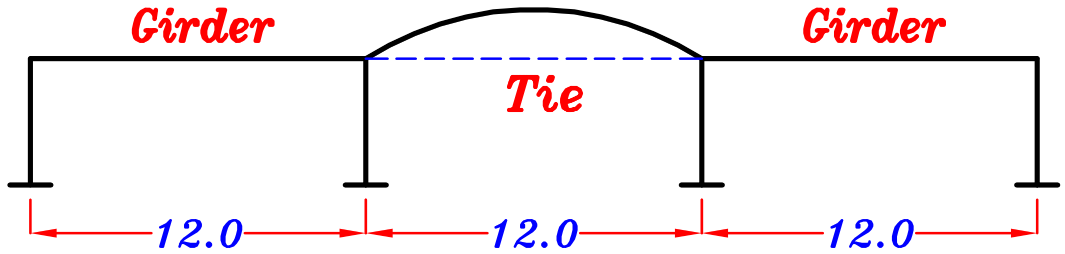
B.M.D.

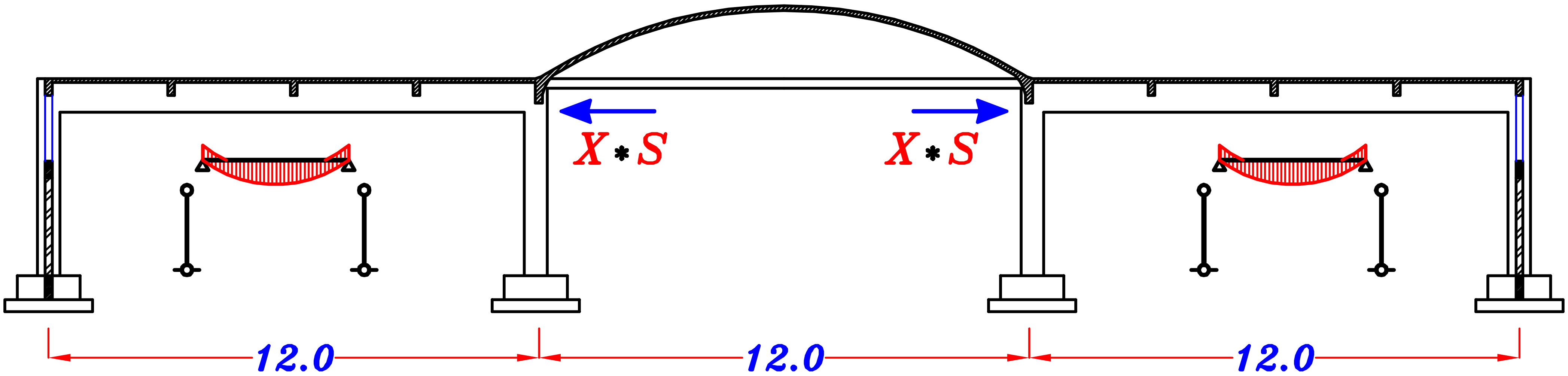


N.F.D.

Example.

With Tie

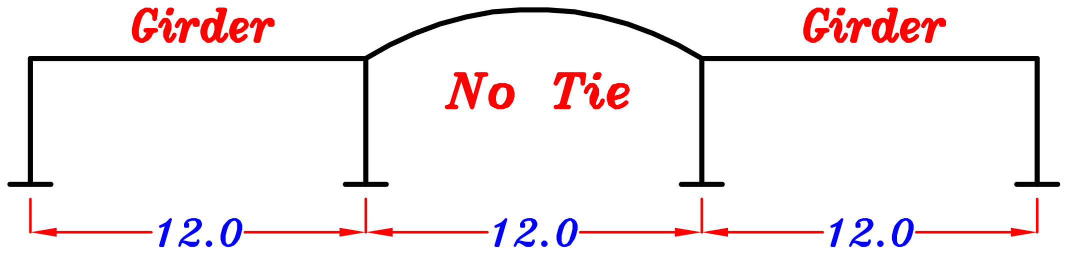




ال ($X*S$) تذهب فقط على ال *Tie* و لا تؤثر على الاعمده المجاوره .

Example.

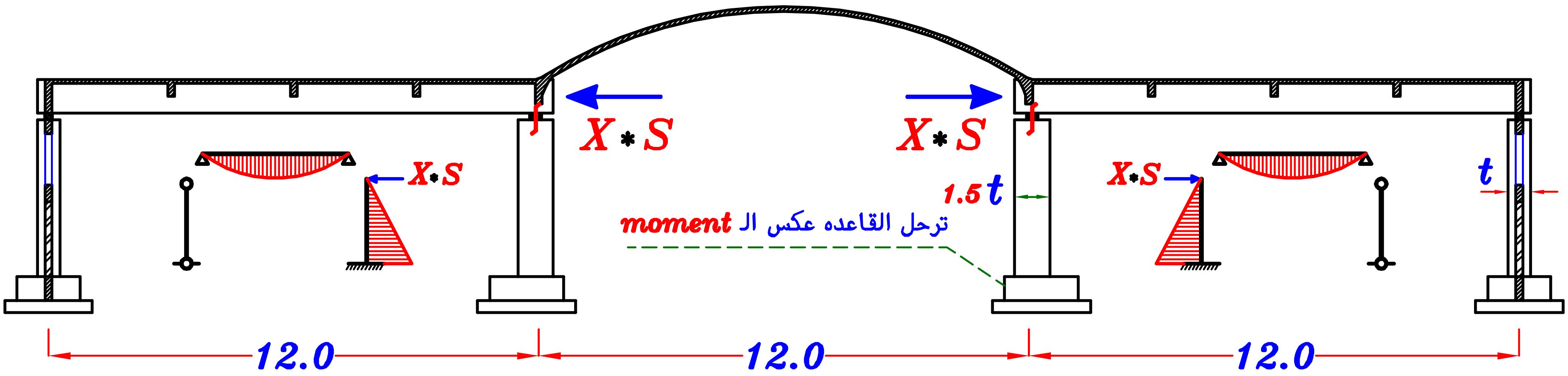
Without Tie



ال $(X*S)$ تنتقل الى ال **Girder**

و لكى يتحمل الحمل الافقى من جهه واحده نعمل على ان ينتقل الحمل الافقى الى عمود واحد فقط و منه الى القاعده مباشره .

فلا يؤثر بأى أحمال أو عزوم اضافيه على العمود الاخر أو كمره ال **Girder** .



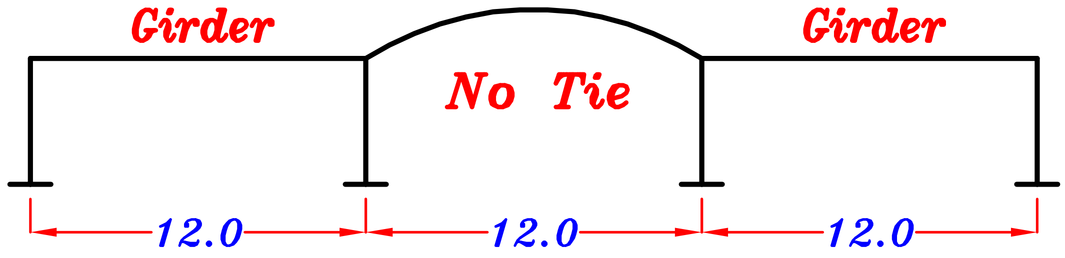
ممكن عمل **Real supports** لل **Girder** ناحيه **Real Hinge** و الناحيه الاخرى **Real Roller**

فينتقل الحمل الافقى كله الى العمود الذى عنده **Real Hinge** و منه الى الارض مباشره .

فنعمل على زياده تخانه هذا العمود (حوالى $1.5\,t$) حتى يتحمل العزوم المؤثره عليه .

Another Solution.

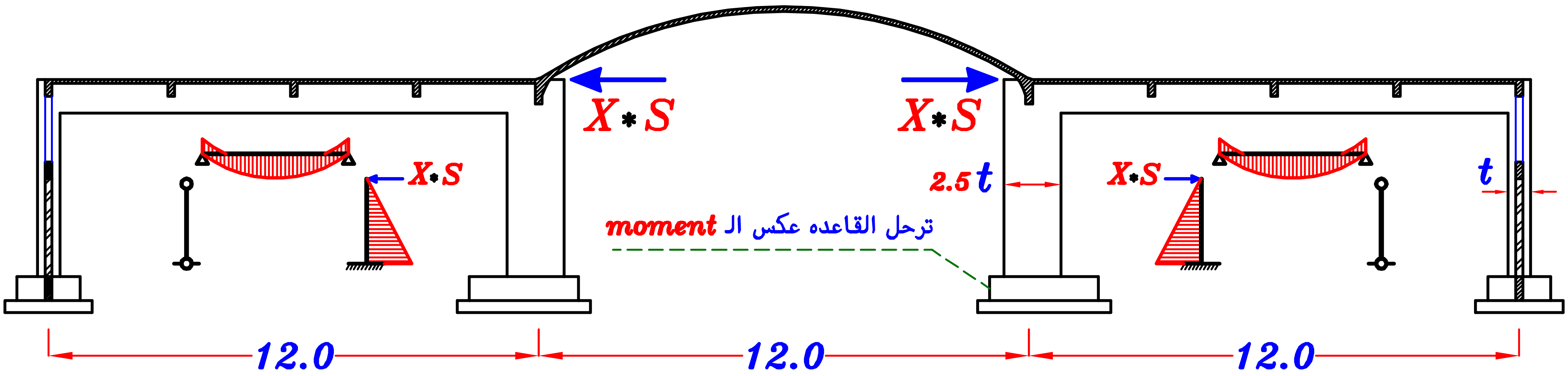
Without Tie



ال $(X * S)$ تنتقل الى ال **Girder**

و لكى يتحمل الحمل الافقى من جهه واحده نعمل على ان ينتقل الحمل الافقى الى عمود واحد فقط و منه الى القاعده مباشره .

فلا يؤثر بأى أحمال أو عزوم اضافيه على العمود الاخر أو كمره ال **Girder** .

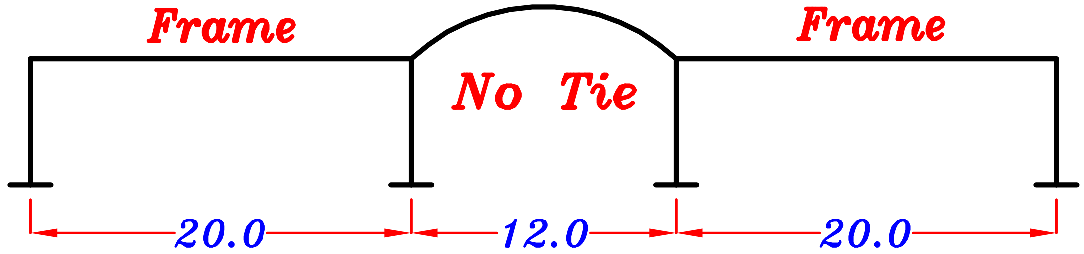


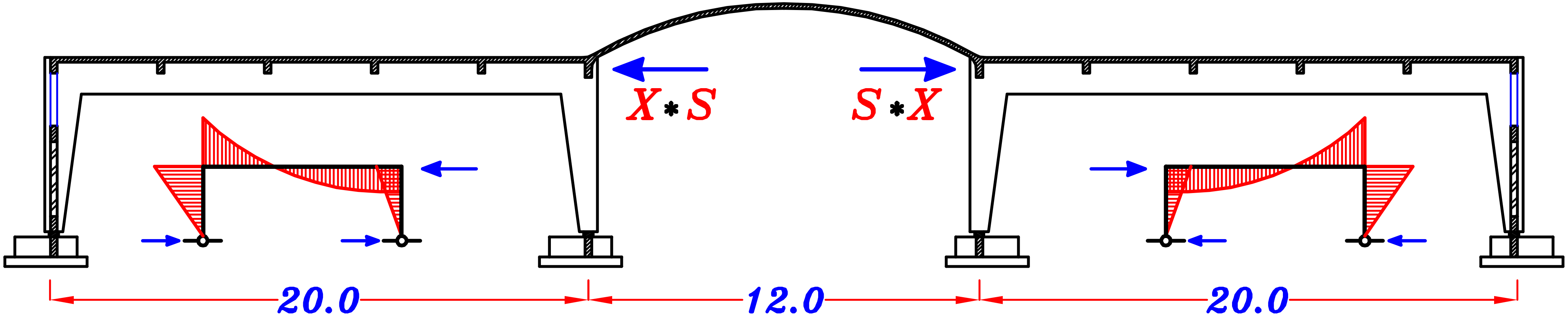
ممكن زياده تخانه عمود من العمودين بقيمه كبيره (حوالى $2.5\,t$) حتى يكون هناك فرق كبير فى ال **stiffness** بين العمودين

فينتقل الحمل الافقى كله الى العمود ذو التخانه الاكبر و منه الى الارض مباشره .

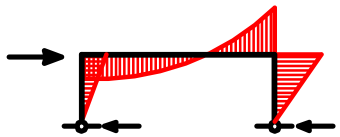
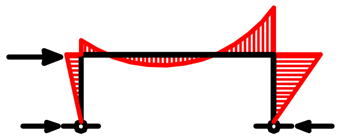
Example.

Without Tie





ال $(X*S)$ تنتقل الى ال **Frame** فتعمل **sway** على ال **Frame**

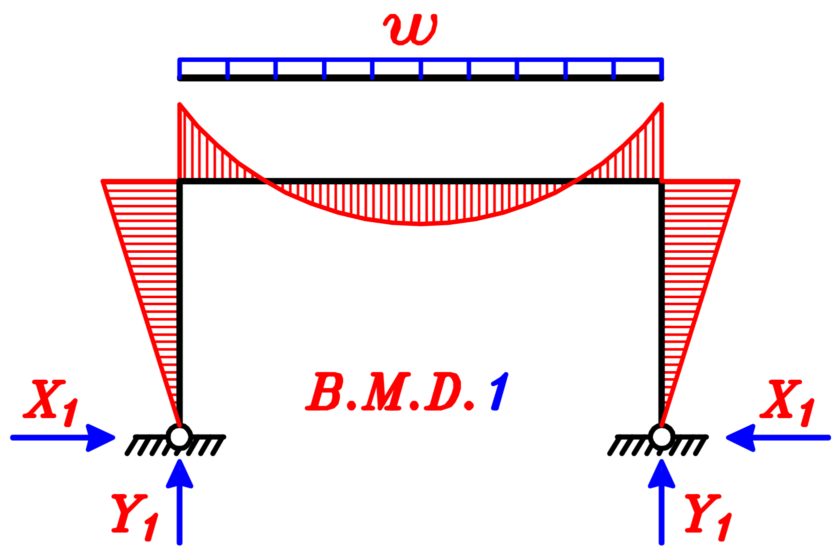
و للاسف لن نستطيع توقع شكل ال **moment**  OR  الا بعد الحل

و يحل ال **Frame** بطريقه ال **Virtual work** او بطريقه ال **Super Position**

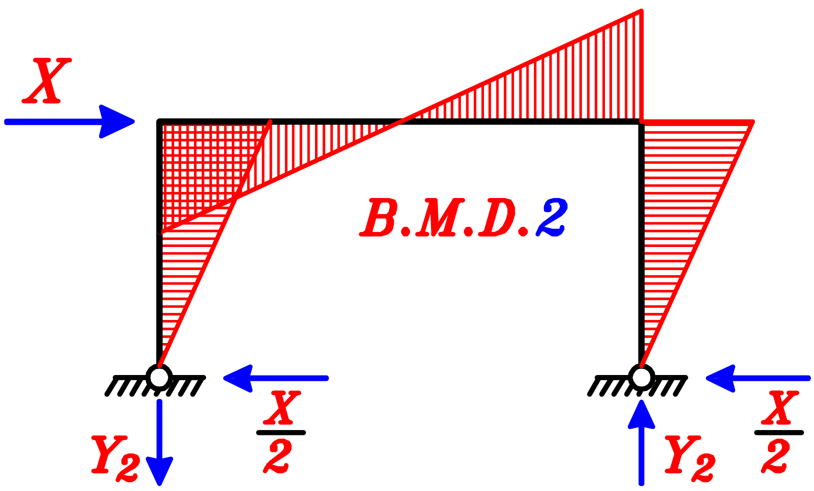
Super Position.

1 – Get B.M. & Reactions due to VL. Load only.

using Moment Distribution
or Approximate Method (IF No Time)

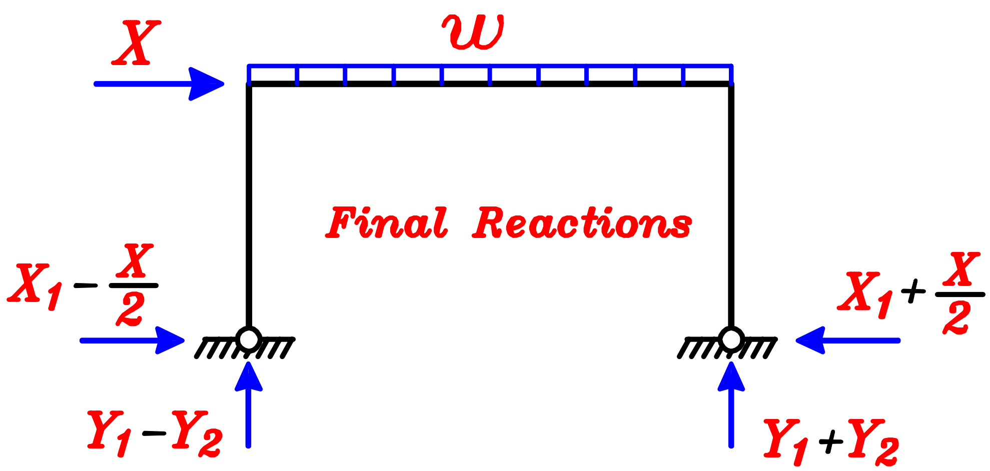


2 – Get B.M. & Reactions due to HL. Load only.



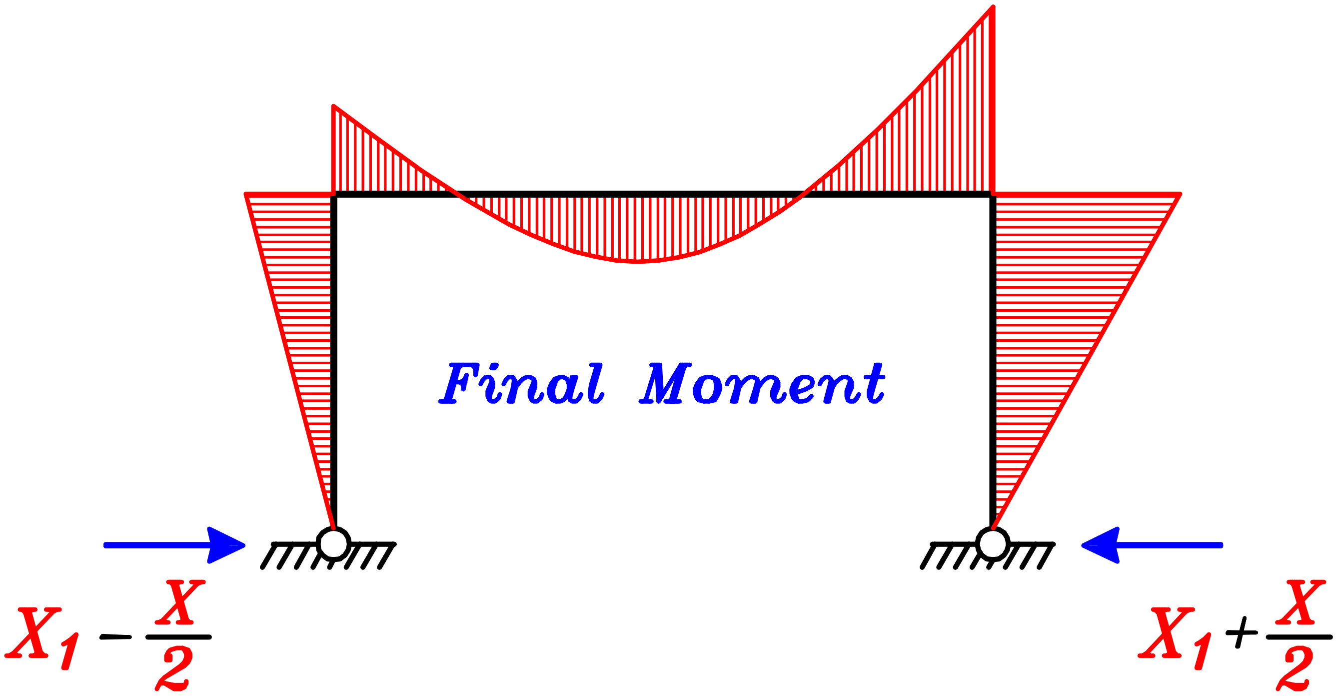
3 – Make Super Position.

يفضل جمع ال Reactions ثم رسم ال B.M.D. & N.F.D.

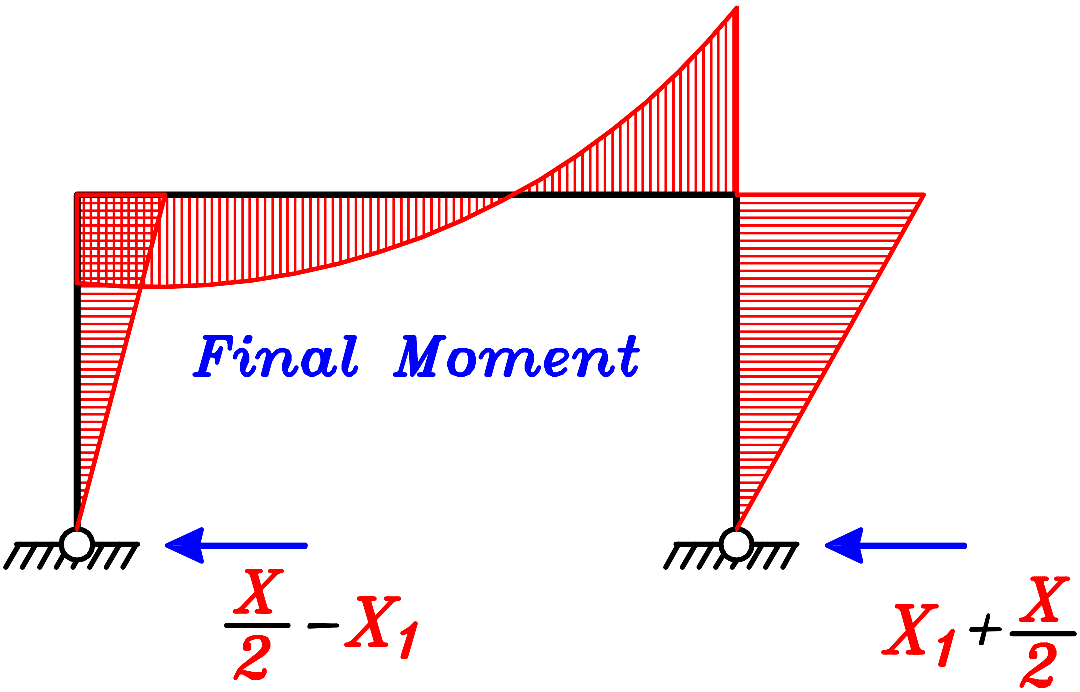


سينتج من ال *Super Position* حاله من حالتين :

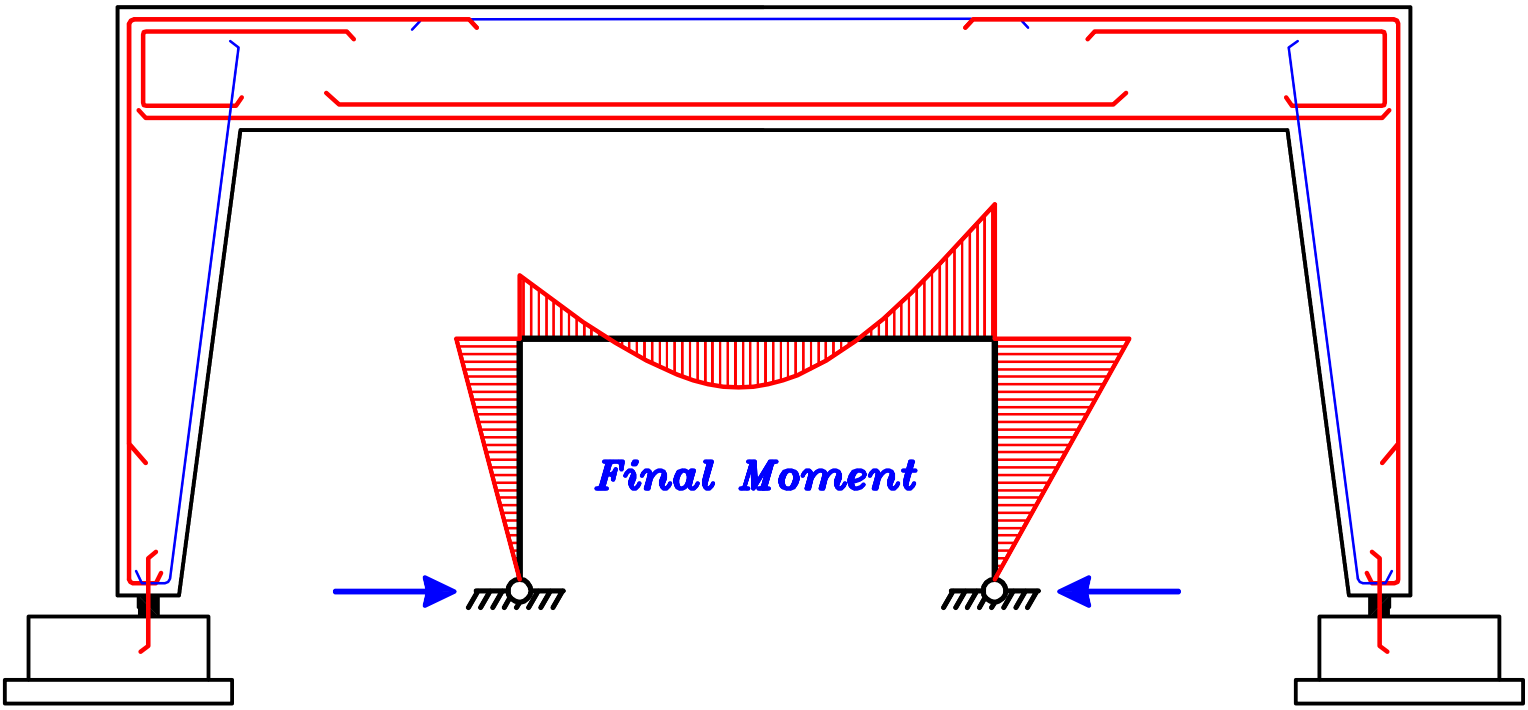
١ ـ أن تكون $X_1 > \frac{X}{2}$ (الحاله الاكثر شيوعاً)

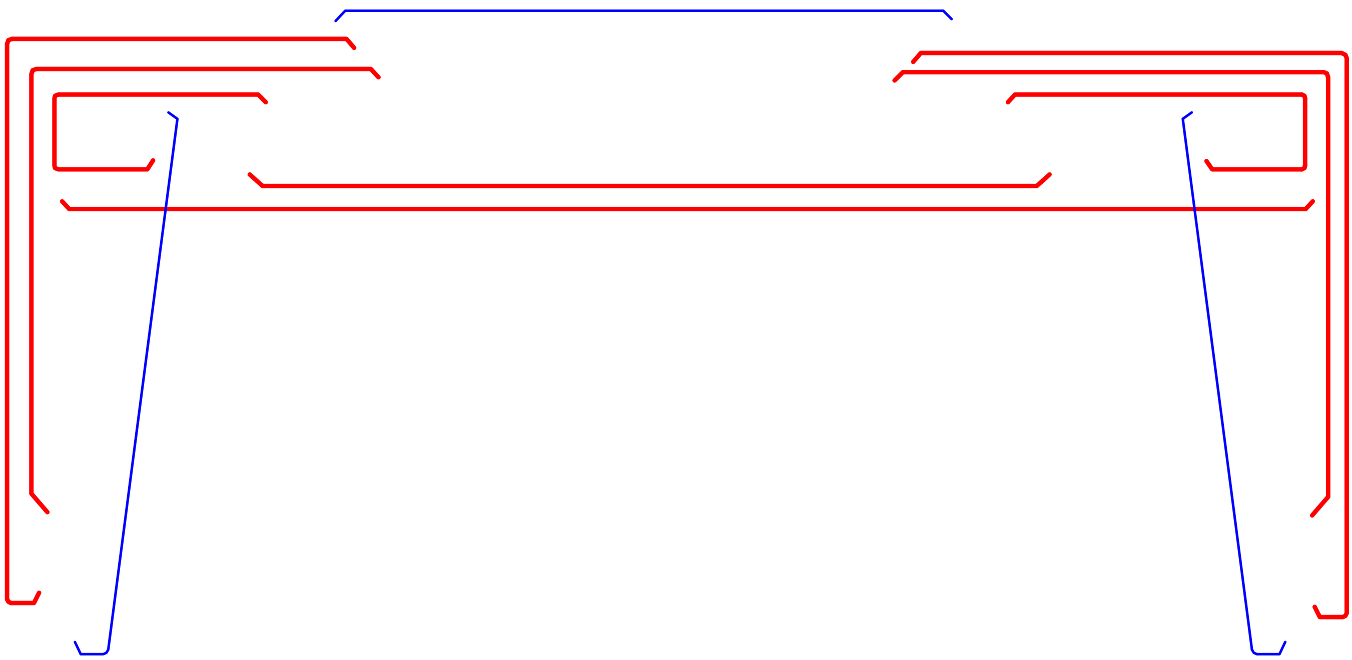


٢ ـ أن تكون $X_1 < \frac{X}{2}$ (الاقل الاكثر شيوعاً)

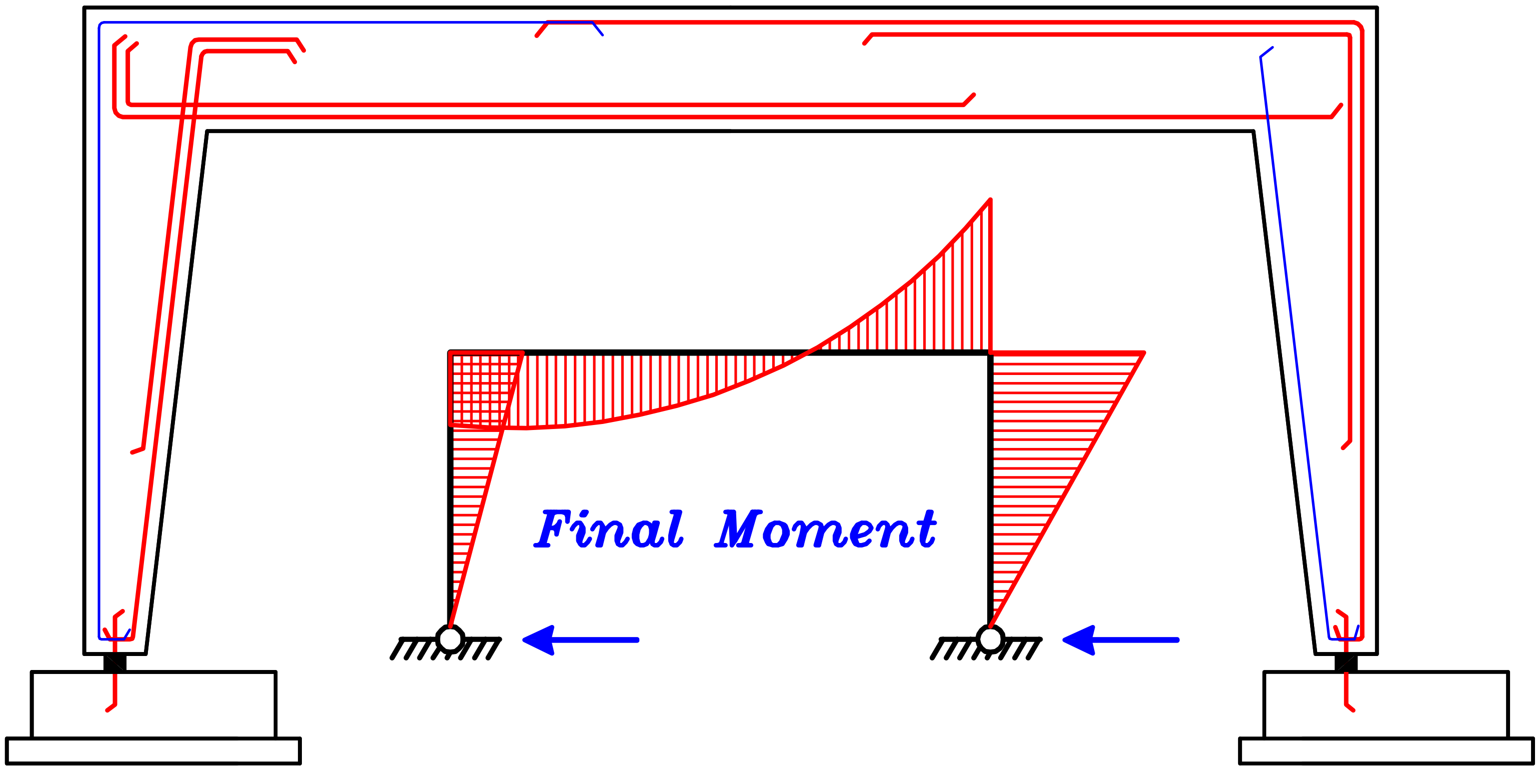


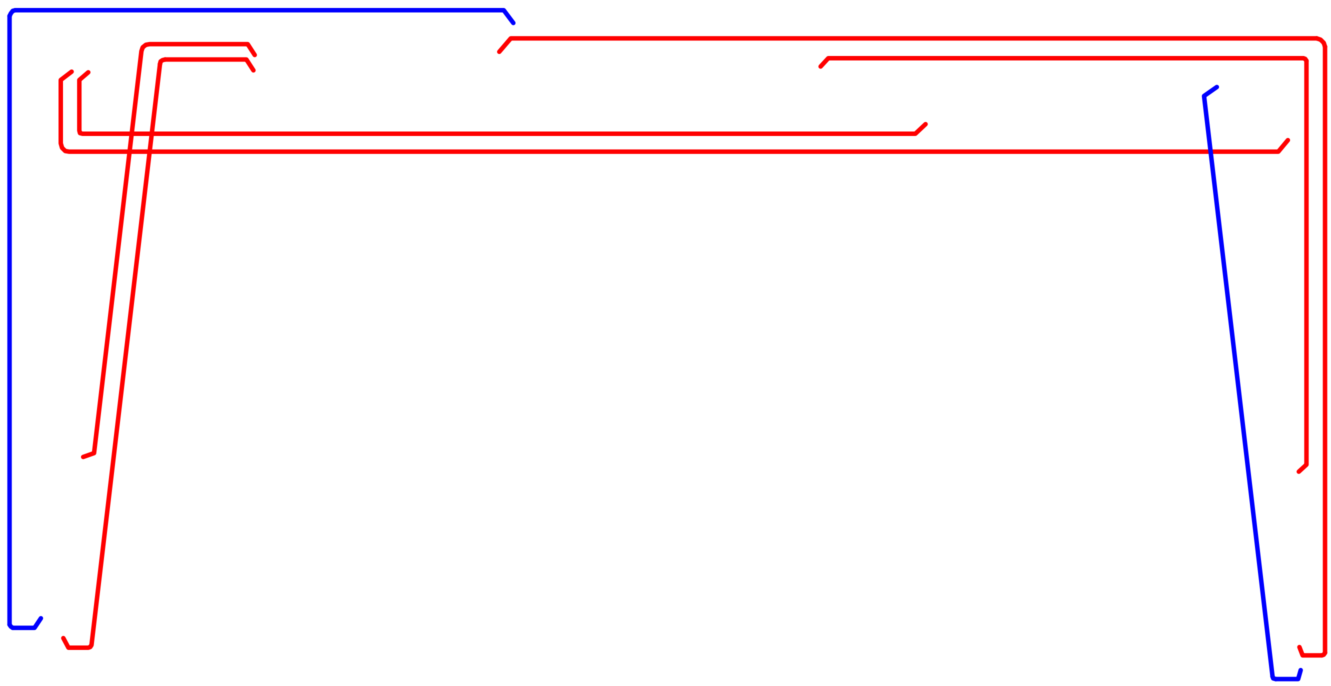
١ ـ اذا كانت $X_1 > \frac{X}{2}$

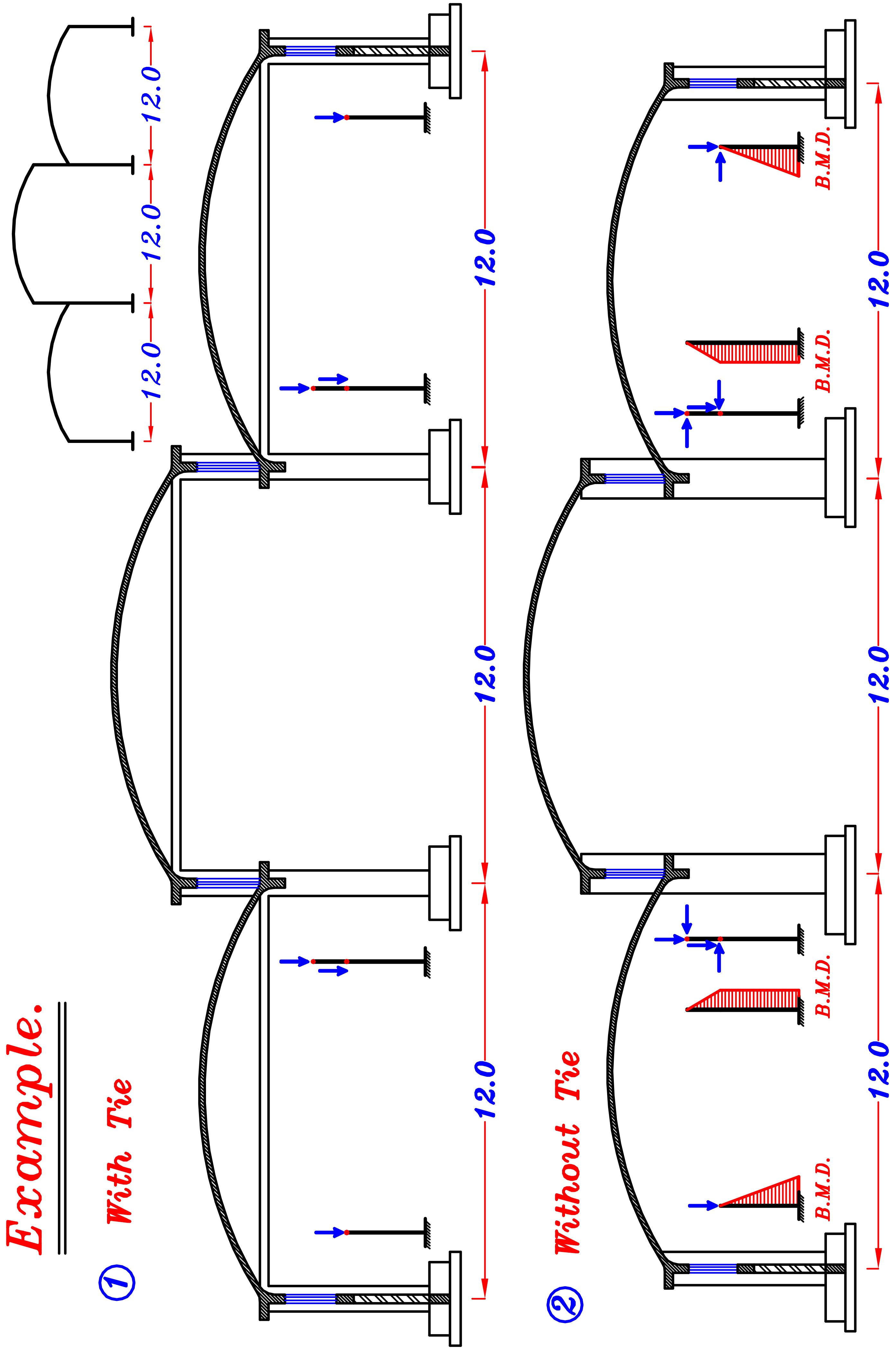


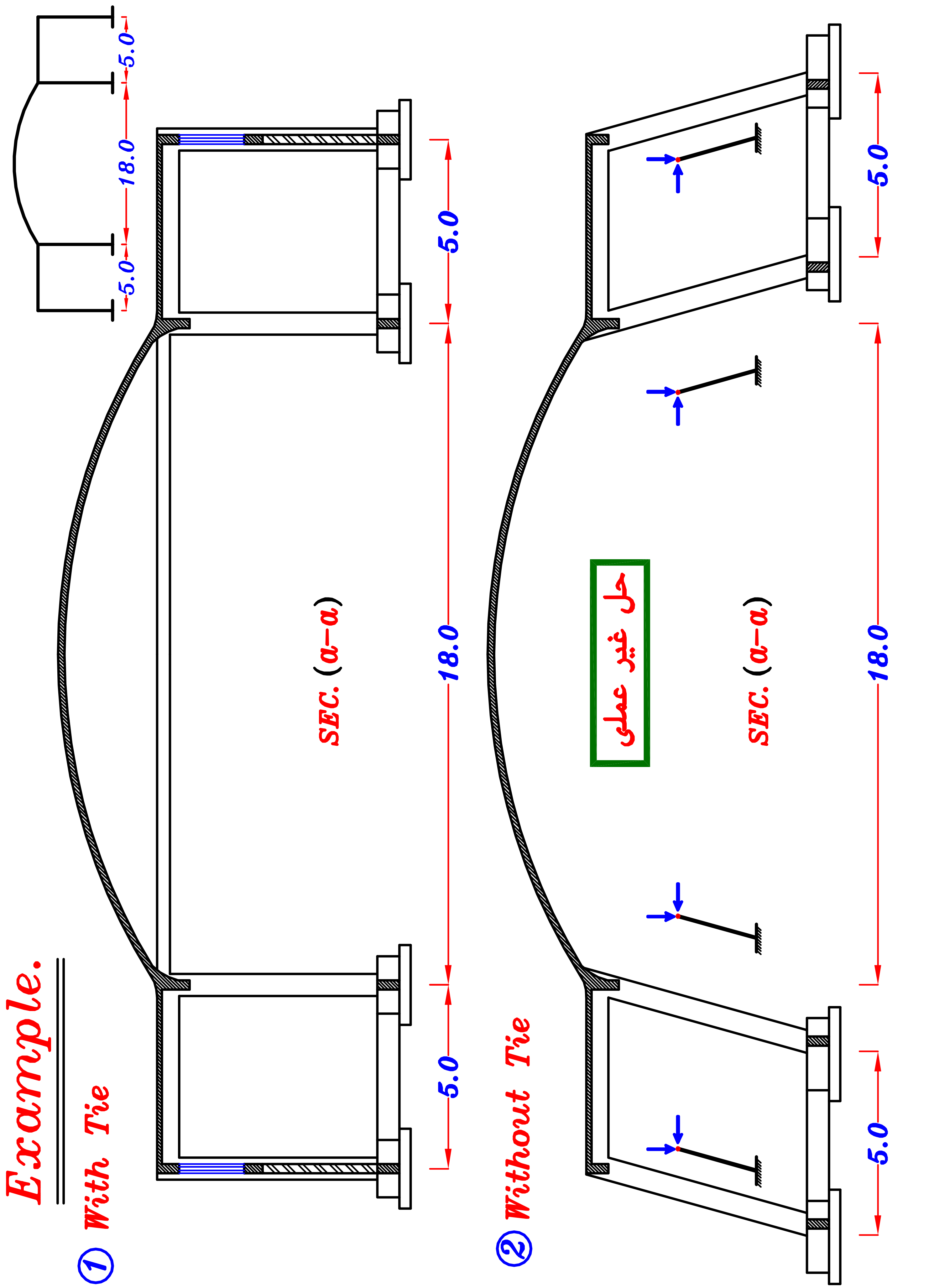


٢ـ اذا كانت $X_1 < \frac{X}{2}$






Example.
① With Tie
② Without Tie
12.0
12.0
12.0
B.M.D.
B.M.D.
B.M.D.
B.M.D.


Example.
① With Tie
② Without Tie
SEC. (a–a)
SEC. (a–a)
5.0
18.0
5.0
حل غير عملى

Arch Slab Examples.

Example.

$F_{cu.} = 25$ N\mm²

$F_y = 360$ N\mm²

L.L. = 0.50 kN\m² H.P.

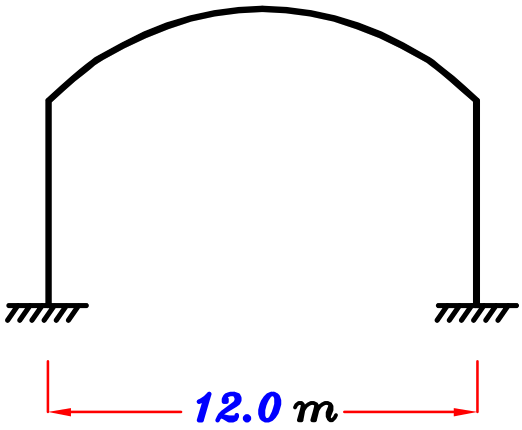
F.C. = 0.50 kN\m² H.P.

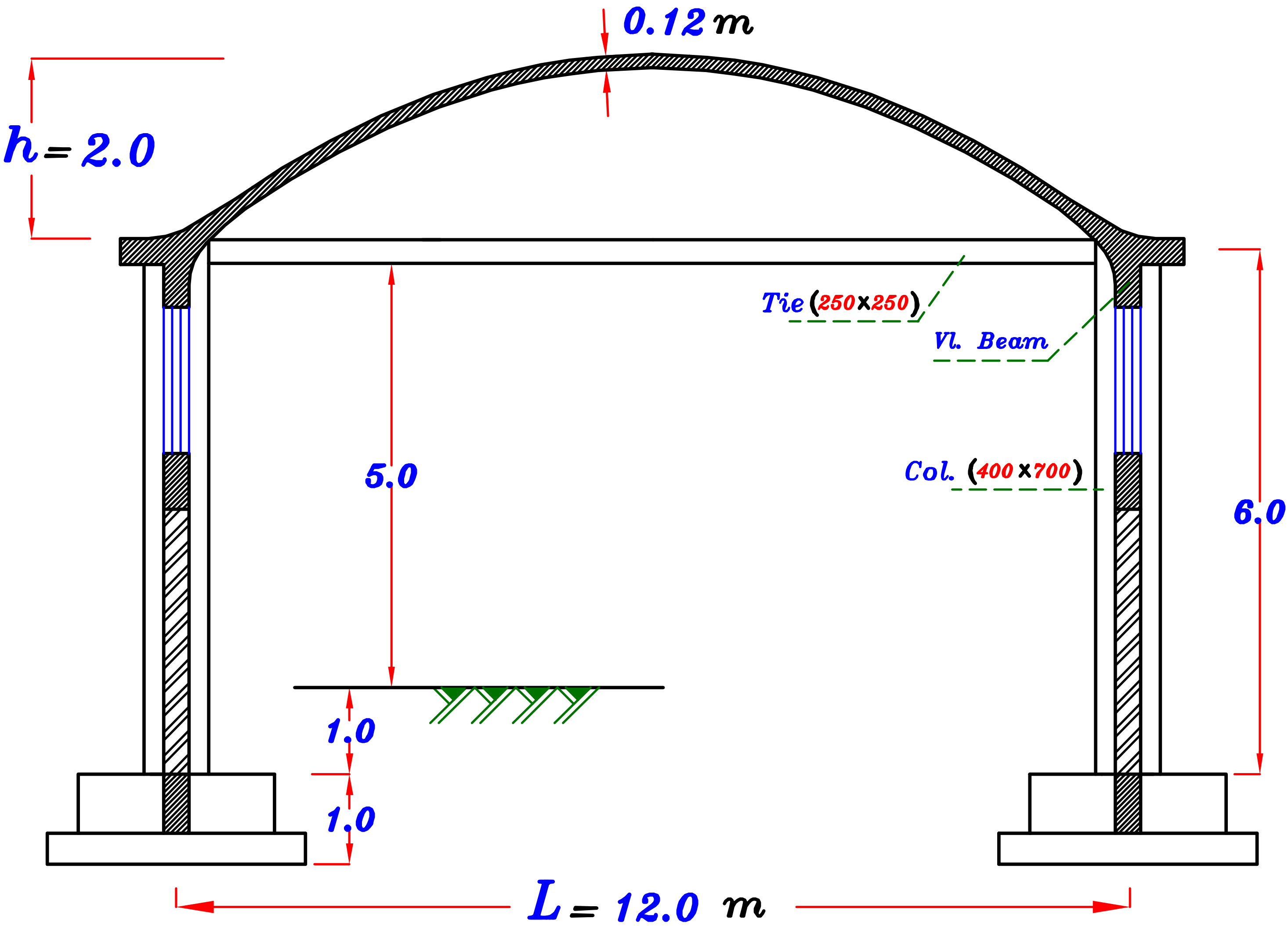
Clear hieght = 5.0 m

Foundation Level. = – 2.0 m

Window hieght = 1.5 m

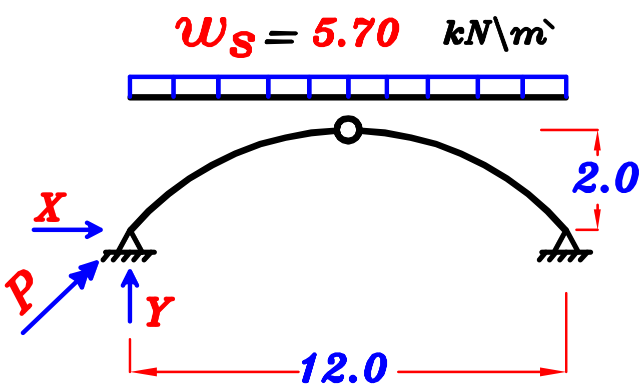
Spacing between Columns = 6.0 m





Design the Arch Slab.

Take $t_s = 120$ mm



$(w_s)_{U.L.} = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)$

$(w_s)_{U.L.} = 1.4\,(0.12 * 25 + 0.50) + 1.6\,(0.50)$

$= 5.70$ kN\m^2 (H.P.)

To Get N.F.

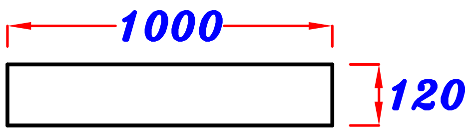
$$Y = \frac{w\,L}{2} = \frac{5.70 * 12}{2} = 34.2 \text{ kN\m}$$

$$X = \frac{w\,L^2}{8\,h} = \frac{5.70 * 12^2}{8 * 2.0} = 51.3 \text{ kN\m}$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{34.2^2 + 51.3^2} = 61.65 \text{ kN}$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



∴ Designed as a Column.

$\therefore P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$

Take $A_c = 120 * 1000 = 120000$ mm^2

$\therefore 61.65 * 10^3 = 0.35\,(120000)(25) + 0.67\,A_s\,(360)$

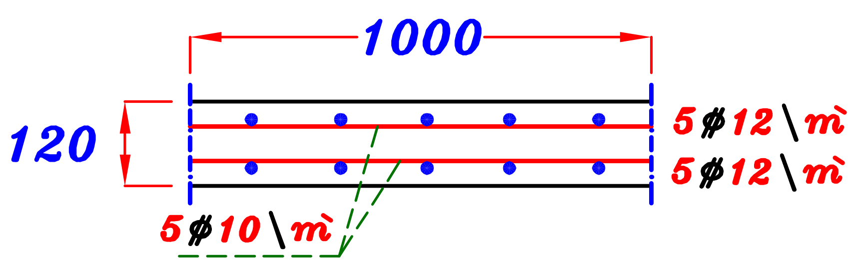
$\therefore A_s = -\,4097$ mm^2 $= -$ (Ve) Value

$\therefore$ Take $A_s = A_{s\,min.} = \frac{0.8}{100} * b * t$

$\therefore A_s = \frac{0.8}{100} * 120 * 1000 = 960$ mm^2 $= A_{s\,total}$

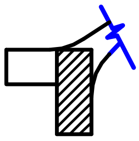
$\therefore$ Upper Steel & Lower Steel $= \frac{A_{s\,total}}{2} = \frac{960}{2} = 480$ mm^2

5ϕ12\m`



Design of End Beam.

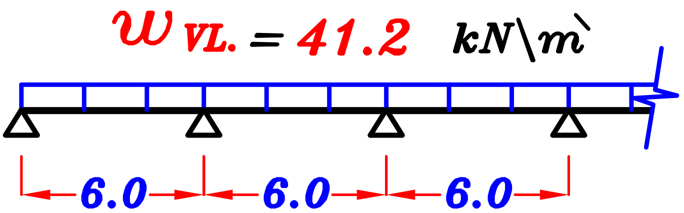
VL. Beam.

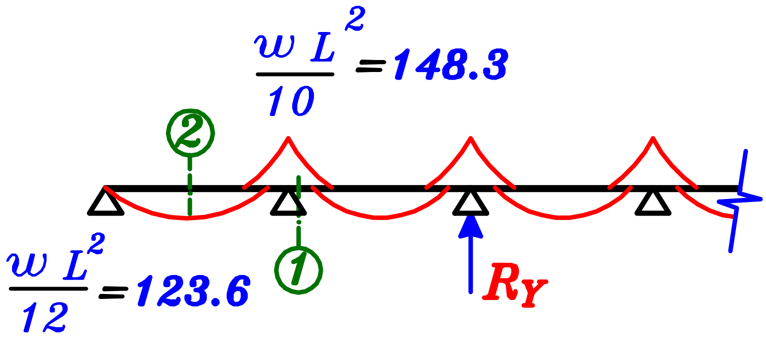


Take O.W. $_{(beam)}$ (VL.+HL.) = 7.0 kN\m` (U.L.)

$$w_{VL.} = O.W._{(beam)} + Y = 7.0 + 34.2 = 41.2 \text{ kN\m`}$$

$$R_Y = w_{VL.} * S = 247.2 \text{ kN}$$





Sec. 1

$M_{U.L.} = 148.3$ kN.m **R–Sec.**

– **Take** $C_1 = 3.50 \longrightarrow J = 0.78$

– **Get** $d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\dfrac{148.3 * 10^6}{25 * 250}} = 539.1$ mm

– **Take** $\boxed{d = 550 \text{ mm}}$, $\boxed{t = 600 \text{ mm}}$

– **Get** $A_s = \dfrac{M_{U.L.}}{J\, F_y\, d} = \dfrac{148.3 * 10^6}{0.78 * 360 * 539.1} = 979.0 \text{ mm}^2$

Check $A_{s\,min.}$ $A_{s_{req.}} = 979.0 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ **Take** $A_s = A_{s_{req.}} = 979.0 \text{ mm}^2$ 5 ϕ 16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \text{ bars}$$

Sec. ② $M_{U.L.} = 123.6$ kN.m

$d = 550$ mm (the same depth of sec. ①)

$$550 = C_1 \sqrt{\frac{123.6 * 10^6}{25 * 250}} \longrightarrow C_1 = 3.91 \longrightarrow J = 0.801$$

$$A_s = \frac{123.6 * 10^6}{0.801 * 360 * 650} = 779.3 \text{ mm}^2$$

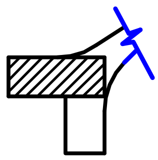
Check $A_{s\,min.}$ $A_{s_{req.}} = 779.3 \text{ mm}^2$

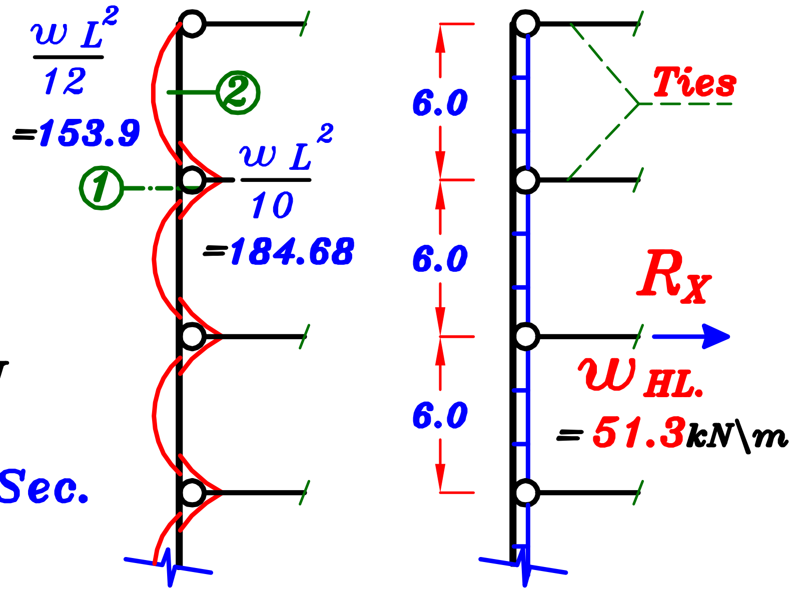
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 779.3 \text{ mm}^2$ $4 \phi 16$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 779.3$ $2 \phi 10$

HL. Beam.





$w_{HL.} = X = 51.3$ kN\m

$R_X = w_{HL.} * S = 307.8$ kN

Design all Sections as R-Sec.

Sec. 1

$M_{U.L.} = 184.68$ kN.m R–Sec.

- Take $C_1 = 3.50 \longrightarrow J = 0.78$

- Get $d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\dfrac{184.68 * 10^6}{25 * 250}} = 601.6$ mm

- Take $\boxed{d = 650 \text{ mm}}$, $\boxed{t = 700 \text{ mm}}$

- Get $A_s = \dfrac{M_{U.L.}}{J\, F_y\, d} = \dfrac{184.68 * 10^6}{0.78 * 360 * 601.6} = 1093.2 \text{ mm}^2$

Check $A_{s\,min.}$ $A_{s_{req.}} = 1093.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1093.2 \text{ mm}^2$ $5 \phi 18$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \text{ bars}$$

Sec. 2

$M_{U.L.} = 153.9$ kN.m

$d = 650$ mm (the same depth of sec. 1)

$$650 = C_1 \sqrt{\frac{153.9 * 10^6}{25 * 250}} \longrightarrow C_1 = 4.14 \longrightarrow J = 0.808$$

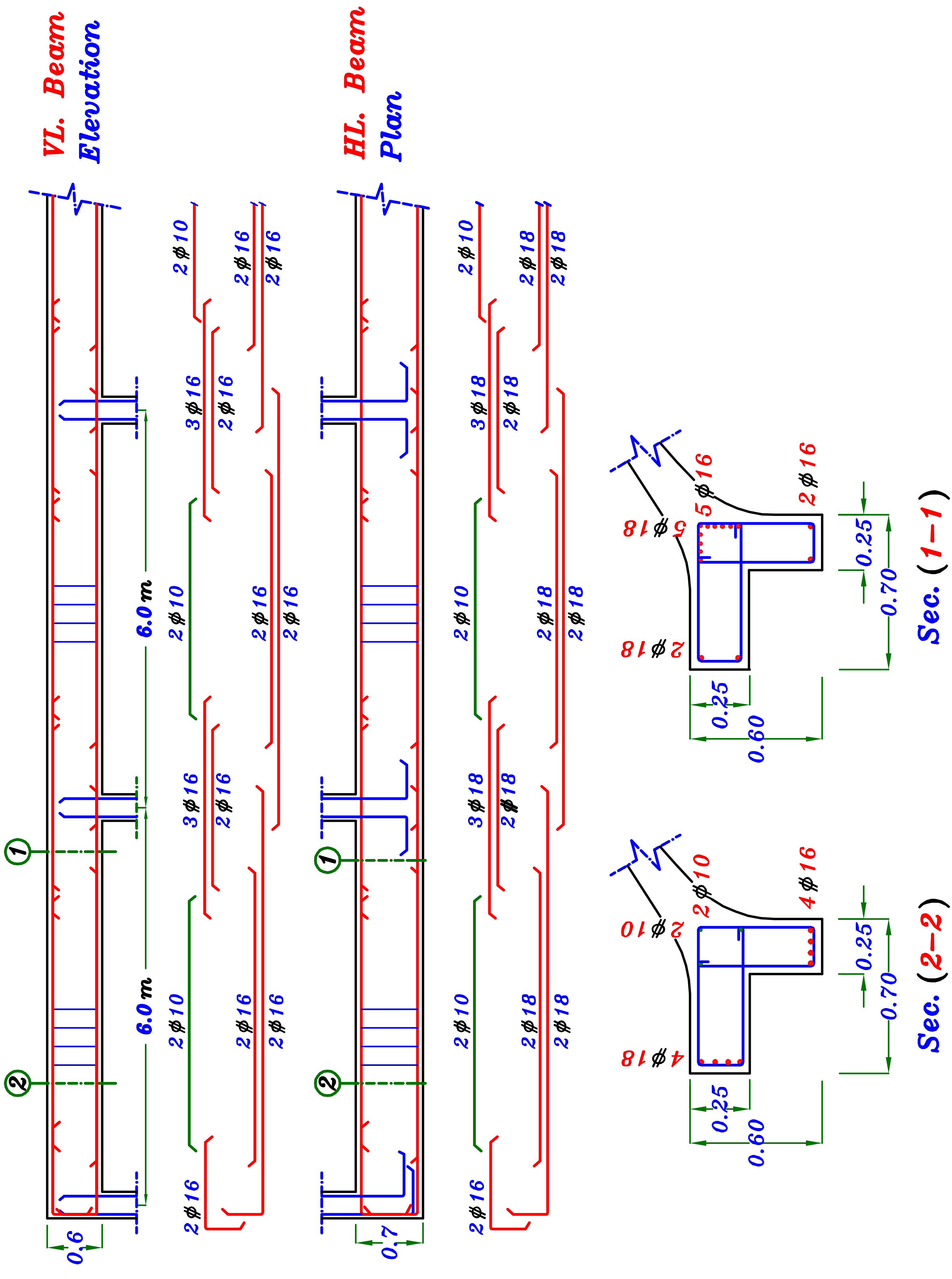
$$A_s = \frac{153.9 * 10^6}{0.808 * 360 * 650} = 813 \text{ mm}^2$$

Check $A_{s\,min.}$ $A_{s_{req.}} = 813 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \text{ mm}^2$$

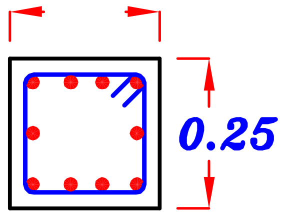
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 813 \text{ mm}^2$ $4 \phi 18$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 813$ $2 \phi 10$


VL. Beam
Elevation
HL. Beam
Plan
2 ϕ 10
2 ϕ 16
2 ϕ 16
3 ϕ 16
2 ϕ 16
6.0 m
2 ϕ 10
2 ϕ 16
2 ϕ 16
3 ϕ 16
2 ϕ 16
6.0 m
2 ϕ 10
2 ϕ 16
2 ϕ 16
2 ϕ 16
0.6
2 ϕ 10
2 ϕ 18
2 ϕ 18
3 ϕ 18
2 ϕ 18
2 ϕ 10
2 ϕ 18
2 ϕ 18
3 ϕ 18
2 ϕ 18
2 ϕ 10
2 ϕ 18
2 ϕ 18
2 ϕ 16
0.7
1
2
5 ϕ 16
2 ϕ 16
5 ϕ 18
2 ϕ 18
0.25
0.70
0.25
0.60
Sec. (1–1)
2 ϕ 10
4 ϕ 16
2 ϕ 10
4 ϕ 18
0.25
0.70
0.25
0.60
Sec. (2–2)

* Design the Tie. $(250 * 250)$

Neglect O.W. $\therefore$ B.M. $\simeq$ Zero



$$T_{(Tie)} = R_X = 307.8 \ kN$$

$$A_S = \frac{T_{(Tie)}}{F_y \backslash \gamma_S} = \frac{307.8 * 10^3}{360 \backslash 1.15} = 983 \ mm^2$$

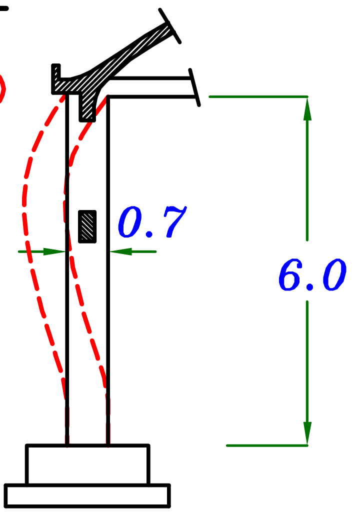
10 ∅12

* Design the Column. $(250 * 700)$

$$N.F. = R_Y = 247.2 \ kN$$

Check Buckling.

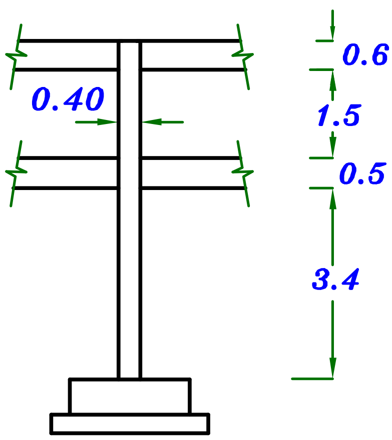
① In plane.



$$H_o = 6.0 \ m$$

$$\lambda_b = \frac{K * H_o}{t} = \frac{1.3 * 6.0}{0.7} = 11.14 > 10$$

② Out of plane.



$$H_o = 3.4 \ m$$

$$\lambda_b = \frac{K * H_o}{b} = \frac{1.2 * 3.4}{0.40} = 10.2 > 10$$

$$\delta = \frac{(\lambda_b)^2 * t}{2000} = \frac{11.14^2 * 0.70}{2000} = 0.043 \ m$$

$$M_{add.} = P * \delta = 247.2 * 0.043 = 10.62 \ kN.m$$

$$e = \frac{M}{P} = \frac{10.62}{247.2} = 0.043\ m \quad \therefore \frac{e}{t} = \frac{0.043}{0.70} = 0.061\ m < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{0.7-0.1}{0.7} = 0.80 \xrightarrow{use} \text{Tables Page } 4\text{–}24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{247.2 * 10^3}{25*400*700} = 0.035 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{10.62 * 10^6}{25*400*700^2} = 0.0021 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

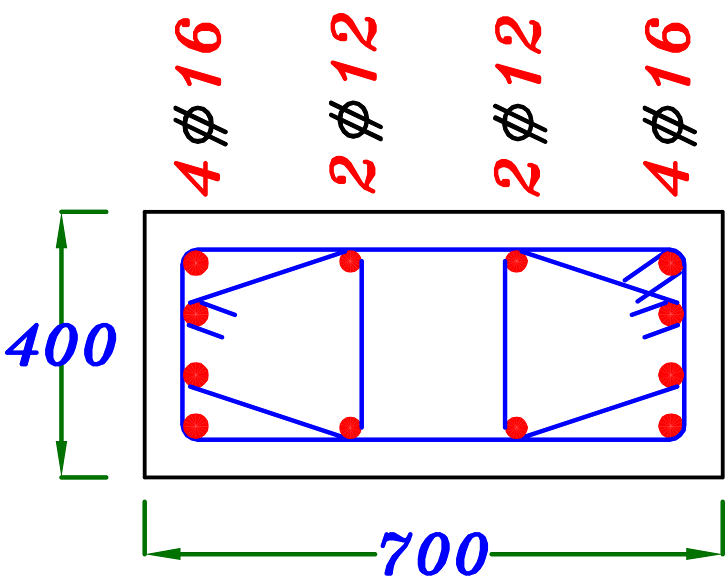
$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 400 * 700 = 700\ mm^2$$

$$A_{s\ total} = A_s + A_s` = 1400\ mm^2$$

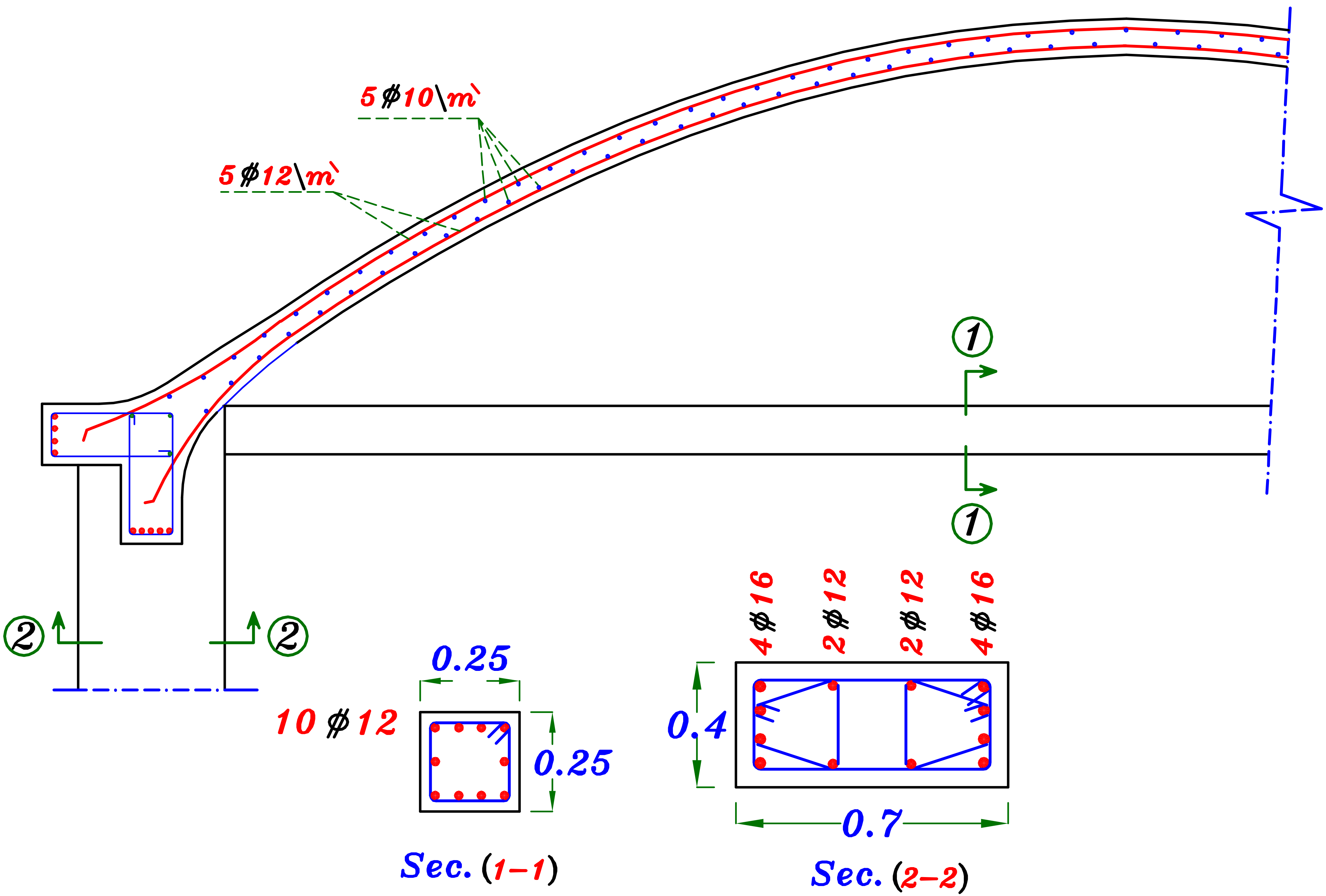
$$A_{s_{min}} = \frac{0.25 + 0.052\ \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\ (11.14)}{100} * 250 * 700 = 1451.2\ mm^2$$

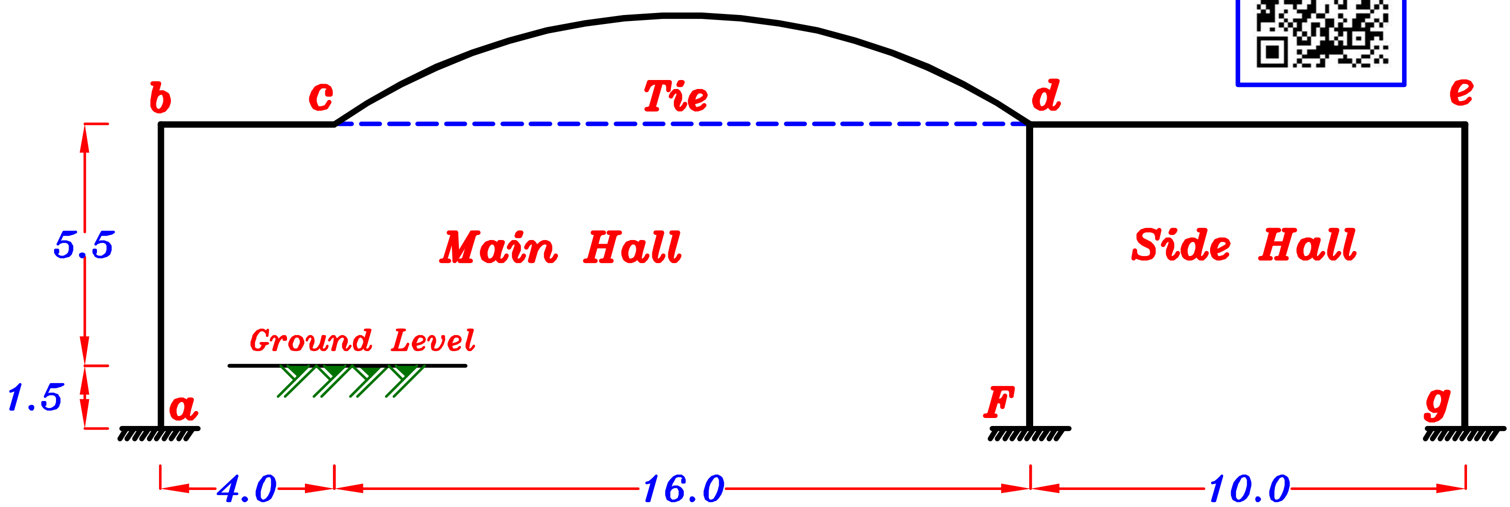
$$A_s = A_s` = \frac{A_{s_{min}}}{2} = \frac{1451.2}{2} = 725.6\ mm^2 \quad 4\phi 16$$



RFT. of the Arch slab.



Example.



Sec. (A–A)

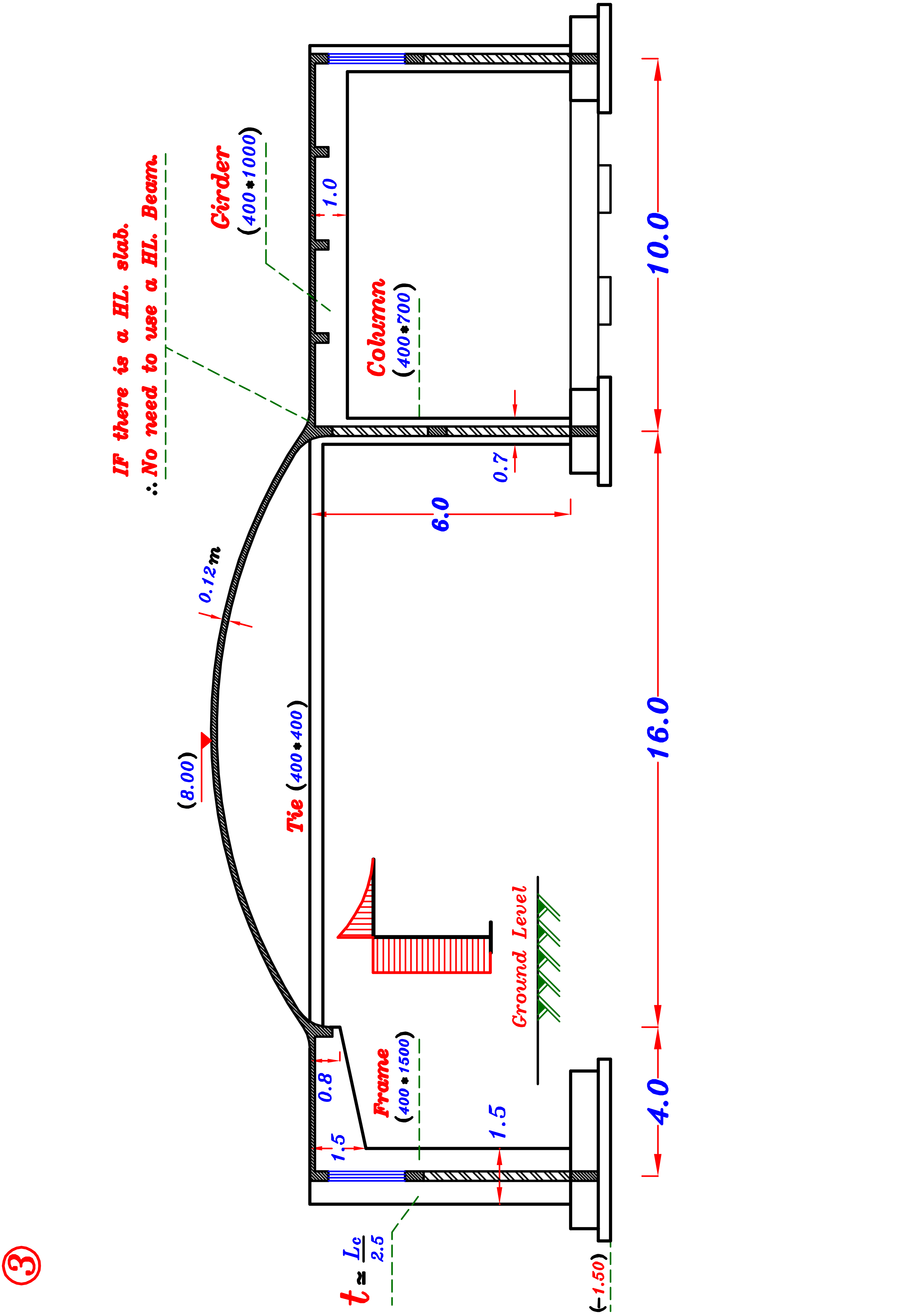
The Fig. shows the general layout of Sec. (A–A) For an Industrial building covering an area (30 * 48 m) .The building consist of a main hall (20 * 48 m) & side hall (10 * 48 m) .
The roof of the main hall is consist of HL. slab & Arched slab with a tie, as shown in the Figure. The side hall is covered with a HL. slab. the columns & the Tie are placed at spacing 6.0 m at the longitudinal direction.
The Foundation level is 1.50 m below the ground level.
Brick walls are 25 cm. thickness are placed along the perimeter between the columns in the longitudinal direction.

Design Data :

* $F_{cu} = 25 \; N \backslash mm^2$ * $F_y = 360 \; N \backslash mm^2$ * $b = 400 \; mm$
* Total loads (D.L. + L.L.) of the HL. slab = 5.0 $kN \backslash m^2$ (H.P.)
* Covering loads (F.C. + L.L.) of the arch slab = 1.0 $kN \backslash m^2$ (H.P.)

Required :

1- Without any calculations, but with reasonably assumed concrete Dim.
Draw a vertical Cross sec. elev. to scale (1:100) For a Full intermediate panel of the main and side halls to show all concrete elements including the Foundations.
2- Design the arched slab. (and all it's supporting elements) & Draw it's Details of reinforcement to scale (1:50)
3- Design the main system a b c & F d e g
& Draw it's Details of reinforcement to scale (1:100)
4- IF the tie c d is removed.
Draw a vertical cross sec. A–A to scale (1:100) For the Full intermediate panel of the main and side halls to show all concrete dimensions including the Foundations.
& design the main systems a b c & F d e g .
& Draw it's Details of reinforcement to scale (1:50)



③
IF there is a HL. slab.
∴ No need to use a HL. Beam.
Girder
(400*1000)
1.0
Column
(400*700)
0.7
6.0
0.12m
(8.00)
Tie (400*400)
Ground Level
Frame
(400*1500)
0.8
1.5
1.5
10.0
16.0
4.0
(-1.50)
t ≈ Lc / 2.5

④ Design the Arch Slab.

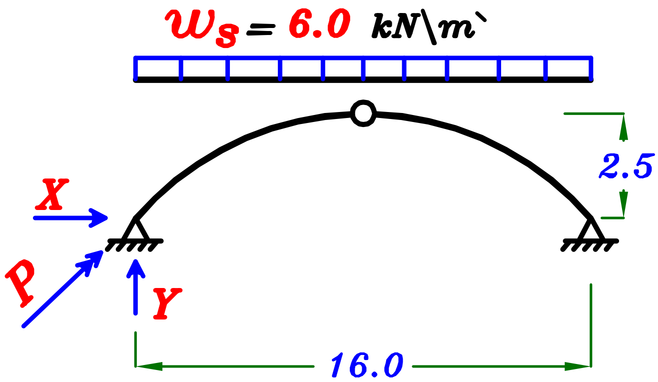
Take $t_s = 120\ mm$

$F.C. + L.L. = 1.0\ kN\backslash m^2$

$(w_s)_{U.L.} = 1.5\,(t_s\,\gamma_c + F.C. + L.L.)$

$(w_s)_{U.L.} = 1.5\,(0.12 * 25 + 1.0)$

$= 6.0\ kN\backslash m^2$ (H.P.)



To Get N.F.

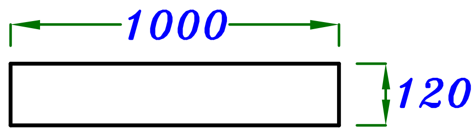
$$Y = \frac{w\,L}{2} = \frac{6.0 * 16}{2} = 48.0\ kN\backslash m$$

$$X = \frac{w\,L^2}{8\,h} = \frac{6.0 * 16^2}{8 * 2.5} = 76.8\ kN\backslash m$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{76.8^2 + 48.0^2} = 90.56\ kN\backslash m$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



$\therefore$ Designed as a Column.

$\therefore\ P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$

Take $A_c = 120 * 1000 = 120000\ mm^2$

$\therefore\ 90.56 * 10^3 = 0.35\,(120000)(25) + 0.67\,A_s\,(360)$

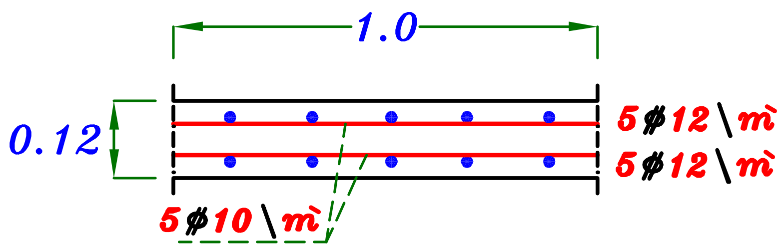
$\therefore\ A_s = -\ 3977\ mm^2 = -$ (Ve) Value

$\therefore$ Take $A_s = A_{s\,min.} = \frac{0.8}{100} * b * t$

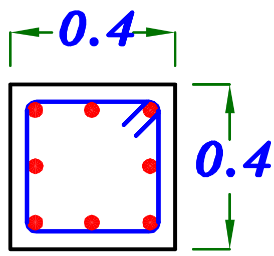
$\therefore\ A_s = \frac{0.8}{100} * 120 * 1000 = 960\ mm^2 = A_{s\,total}$

$\therefore$ Upper Steel & Lower Steel $= \frac{A_{s\,total}}{2} = \frac{960}{2} = 480\ mm^2$

5ϕ12\m`



* Design the Tie. (400 * 400)



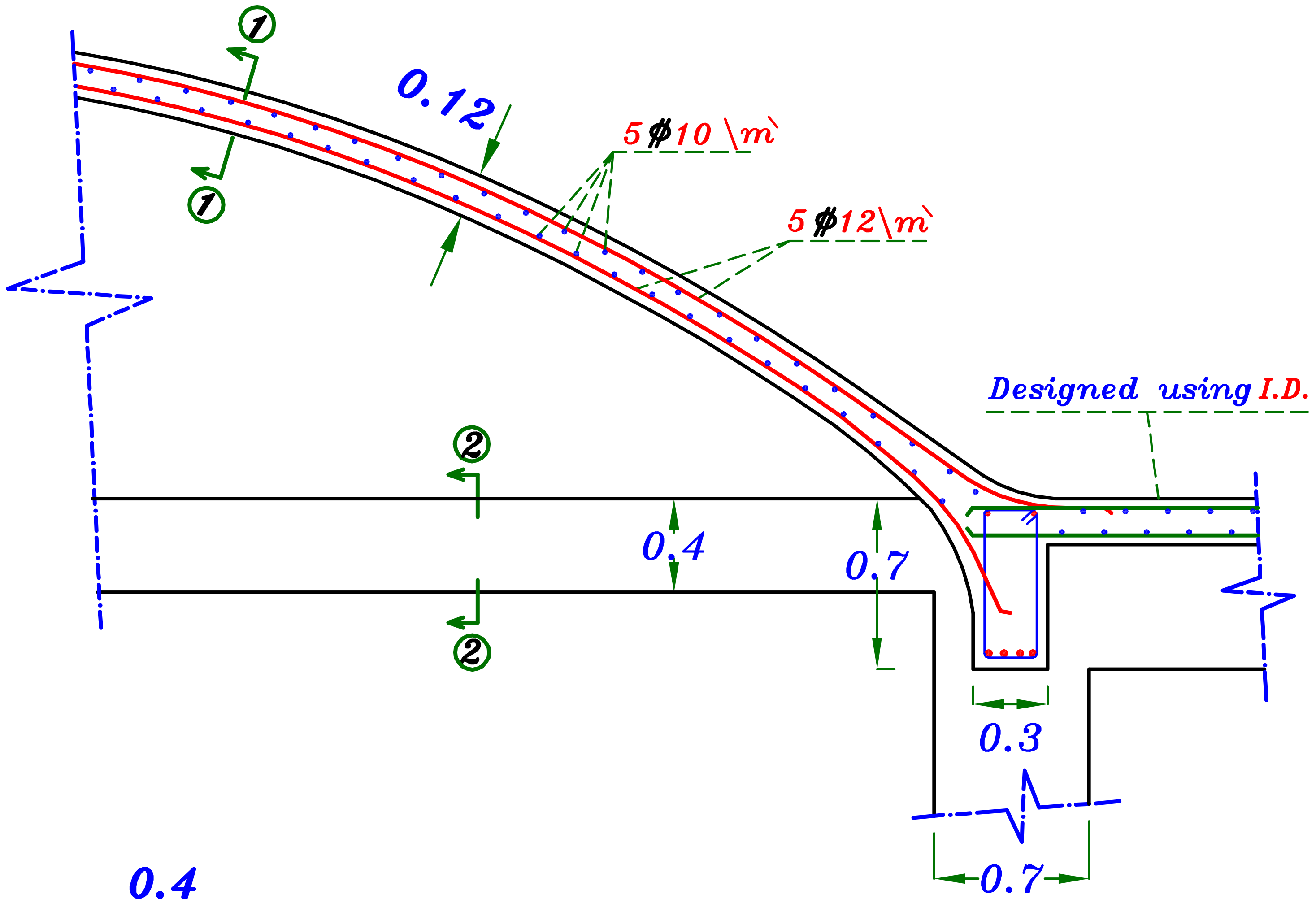
8 ϕ 16

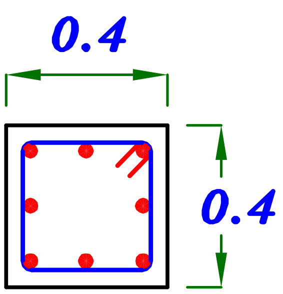
$$T_{(Tie)} = X * S = 76.8 * 6.0 = 460.8 \text{ kN}$$

$$A_s = \frac{T_{(Tie)}}{F_y \backslash \gamma_s} = \frac{460.8 * 10^3}{360 \backslash 1.15} = 1472 \text{ mm}^2$$

8 ϕ 16

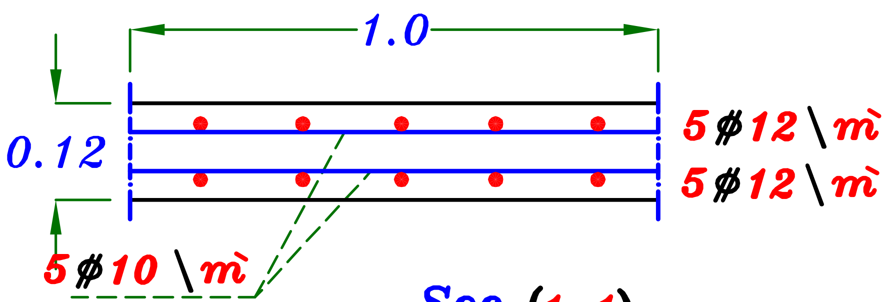
RFT. of the Arch slab.





8 ϕ 16

Sec. (2–2)



Sec. (1–1)

③ Design the main system.

Loads on secondary Beam.

$o.w._{S.beam} = 4.20 \ kN\backslash m'$

$\because$ Total loads on HL. slab $= 5.0 \ kN\backslash m^2$ as given in data.

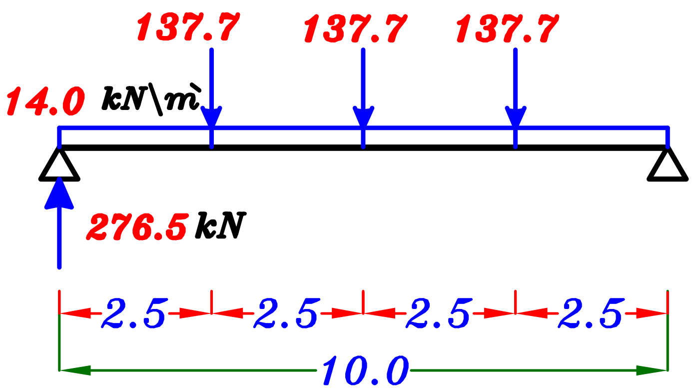
$\therefore w_s = 1.5 * 5.0 = 7.50 \ kN\backslash m^2$

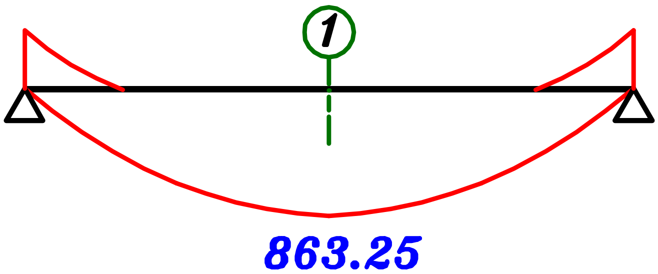
$$w = o.w._{S.beam} + 2\left(w_s \frac{L_s}{2}\right) = 4.2 + 2\,(7.50)\left(\frac{2.5}{2}\right) = 22.95 \ kN\backslash m'$$

$$R = w_a * S = 22.95 * 6.0 = 137.7 \ kN$$

Loads on Girder d e

$o.w._{Girder} = 1.4\,(0.40)\,(1.0)\,(25) = 14.0 \ kN\backslash m'$





Design the Girder. (400*1000)

Sec. ① T−Sec. $M = 863.25 \ kN.m$

$$B = \begin{cases} C.L.-C.L. = 6.0 \ m = 6000 \ mm \\ 16\,t_s + b = 16*120 + 400 = 2320 \ mm \\ K\,\dfrac{L}{5} + b = 1.0 * \dfrac{10000}{5} + 400 = 2400 \ mm \end{cases}$$

$$\boxed{B = 2320 \ mm}$$

$$\therefore 950 = C_1 \sqrt{\frac{863.25*10^6}{25*2320}} \longrightarrow C_1 = 7.79 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{863.25*10^6}{0.826*360*950} = 3055 \ mm^2 \quad 12\phi 18$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 3055 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 950 = 1187.5 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3055 \ mm^2$ $\quad 12\phi18$

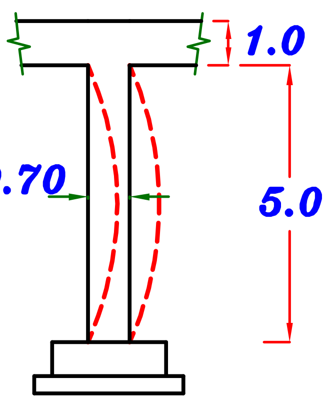
$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{18 + 25} = 8.72 = 8.0 \text{ bars}$$

Design of the Columns. (400*700)

$$P = R_{(Girder)} + R_{(End\ Beam)} = R_{(Girder)} + (o.w. + Y) * S$$

$$= 276.5 + (4.2 + 48) * 6.0 = 589.7 \ kN$$

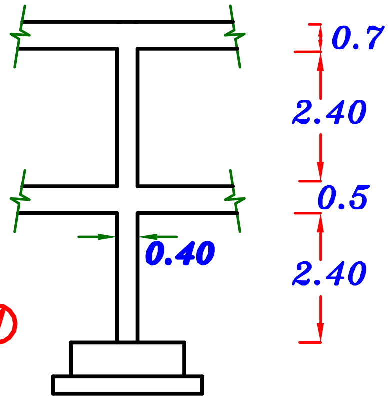
① In Plane.



$H_o = 5.0 \ m$

$$\lambda_b = \frac{K * H_o}{t} = \frac{1.2 * 5.0}{0.7} = 8.57 < 10$$

② Out of Plane.



$H_o = 2.4 \ m$

$$\lambda_b = \frac{K * H_o}{b} = \frac{1.2 * 2.40}{0.40} = 7.2 < 10$$

The column is Short Column. (400*700)

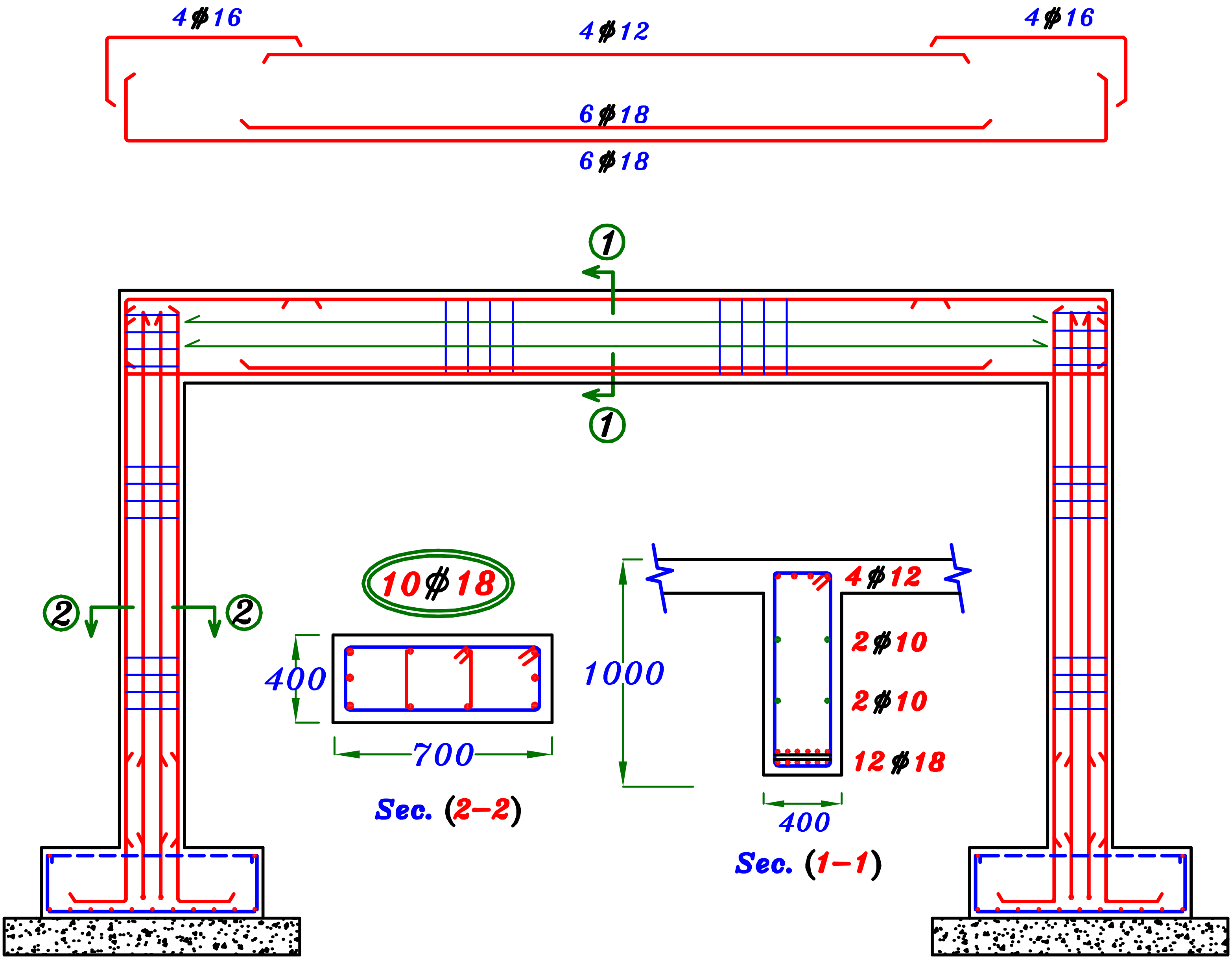
$$\therefore P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$$

$$\therefore 589.7 * 10^3 = 0.35\,(400 * 700)(25) + 0.67\, A_s\,(360)$$

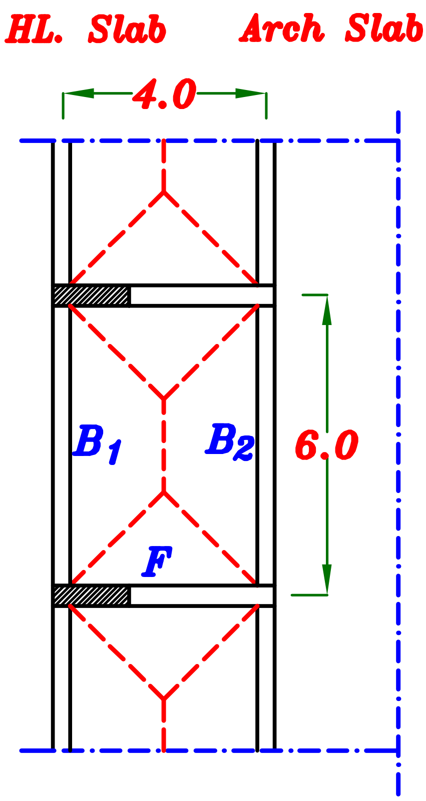
$$\therefore A_s = -7712 \ mm^2 = (-Ve) \text{ Value}$$

$$\therefore \text{Take } A_s = A_{s_{min.}} = \frac{0.8}{100} * b * t$$

$$\therefore A_s = \frac{0.8}{100} * 400 * 700 = 2240 \ mm^2$$ $\quad 10\phi18$


4ϕ16
4ϕ12
4ϕ16
6ϕ18
6ϕ18
1
1
2
2
10ϕ18
400
700
Sec. (2−2)
1000
4ϕ12
2ϕ10
2ϕ10
12ϕ18
400
Sec. (1−1)

© Frame a b c .



Loads on beam. B_1

For Trapezoid $C_a = 1 - \frac{1}{2}\left(\frac{L_s}{L}\right) = 1 - \frac{1}{2}\left(\frac{4.0}{6.0}\right) = \frac{2}{3}$

$$w_a = o.w. + C_a\, w_s\, \frac{L_s}{2}$$

$$= 4.20 + \frac{2}{3}\,(1.5 * 5.0)\left(\frac{4.0}{2}\right) = 14.2 \;\; kN\backslash m`$$

$R_1 = 14.2 * 6.0 = 85.2 \;\; kN$ $\boxed{R_1 = 85.2 \;\; kN}$

Loads on beam. B_2

تحمل الكمره B_2 كلا من البلاطه الافقيه و البلاطه ال Arch

من جهة البلاطه الافقيه تحمل حمل على شكل Trapezium من الجهه الاخرى

اما من جهة البلاطه ال Arch فتحمل قيمه $Y = \frac{w_s\, L}{2} = 48.0$ kN/m

$$w_a = o.w. + C_a\, w_s\, \frac{L_s}{2} \quad \overset{Arch\ Slab}{+\; Y}$$

$$= 4.20 + \frac{2}{3}\,(1.5 * 5.0)\left(\frac{4.0}{2}\right) + 48.0 = 62.2 \;\; kN\backslash m`$$

$R_2 = 62.2 * 6.0 = 373.2 \;\; kN$ $\boxed{R_2 = 373.2 \;\; kN}$

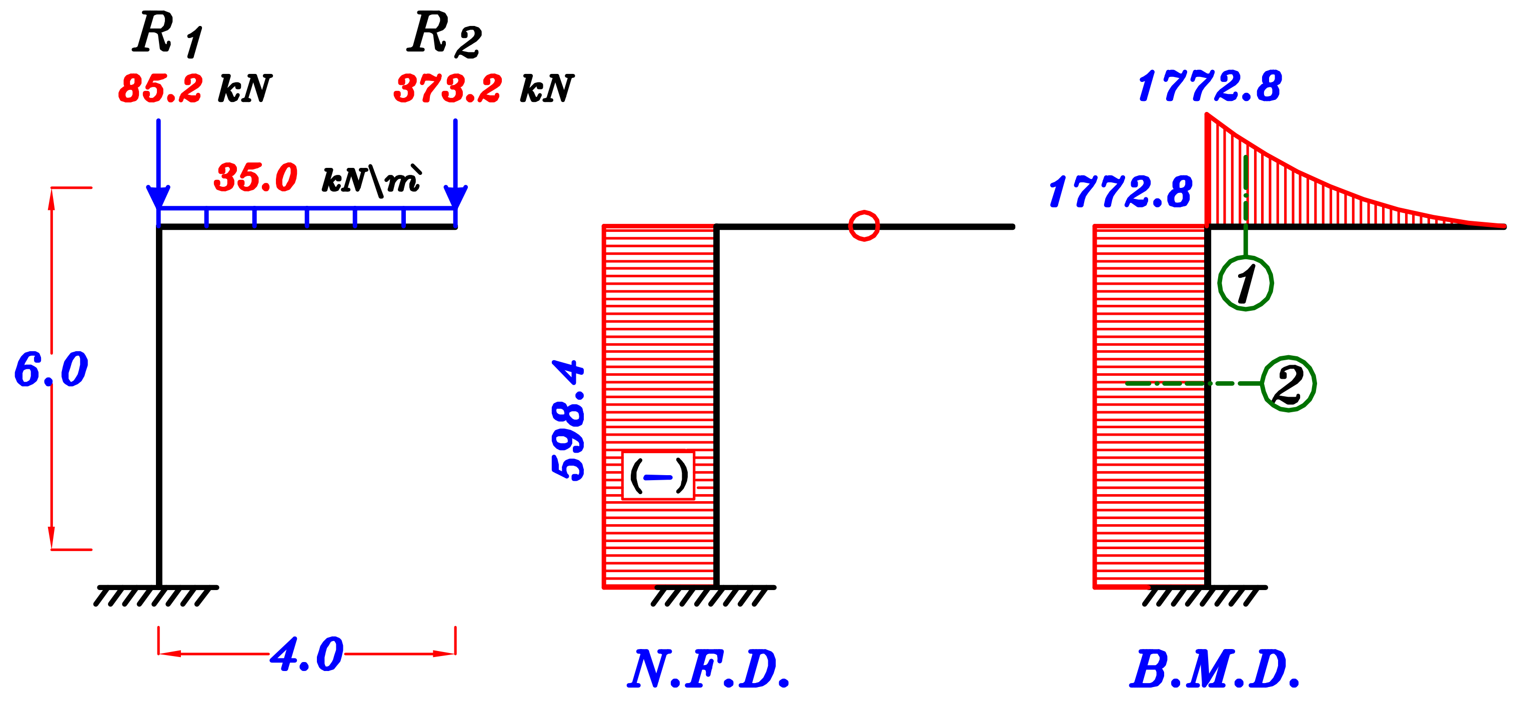
Loads on Frame. F

Take O.W. (Frame) $= 15.0 \;\; kN\backslash m`$ (U.L.)

For Triangle $C_e = \frac{2}{3}$

$$w_2 = o.w. + 2\, C_e\, w_s\, \frac{L_s}{2}$$

$$= 15.0 + 2 * \frac{2}{3}\,(1.5 * 5.0)\left(\frac{4.0}{2}\right) = 35.0 \;\; kN\backslash m`$$



Design of Frame.

Sec. 1 R−sec.

$M = 1772.8$ kN.m , P = zero , $b = 400$ mm , $t = 1500$ mm

$$\therefore 1400 = C_1 \sqrt{\frac{1772.8 * 10^6}{25 * 400}} \longrightarrow C_1 = 3.32 \longrightarrow J = 0.769$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1772.8 * 10^6}{0.769 * 360 * 1400} = 4574 \text{ mm}^2$$

Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 4574 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1400 = 1750 \text{ mm}^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 4574 \text{ mm}^2 \quad 10\phi25$$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

$$\text{Stirrup Hangers} = (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 4574 \quad 5\phi12$$

Sec. ② R-sec.

Neglect Effect of Buckling.

$M = 1772.8$ kN.m , $P = 598.4$ kN , $b = 400$ mm , $t = 1500$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{598.4 * 10^3}{25 * 400 * 1500} = 0.04$ (We can neglect P)

$\therefore 1400 = C_1 \sqrt{\frac{1772.8 * 10^6}{25 * 400}} \longrightarrow C_1 = 3.32 \longrightarrow J = 0.769$

$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1772.8 * 10^6}{0.769 * 360 * 1400} = 4574 \; mm^2$

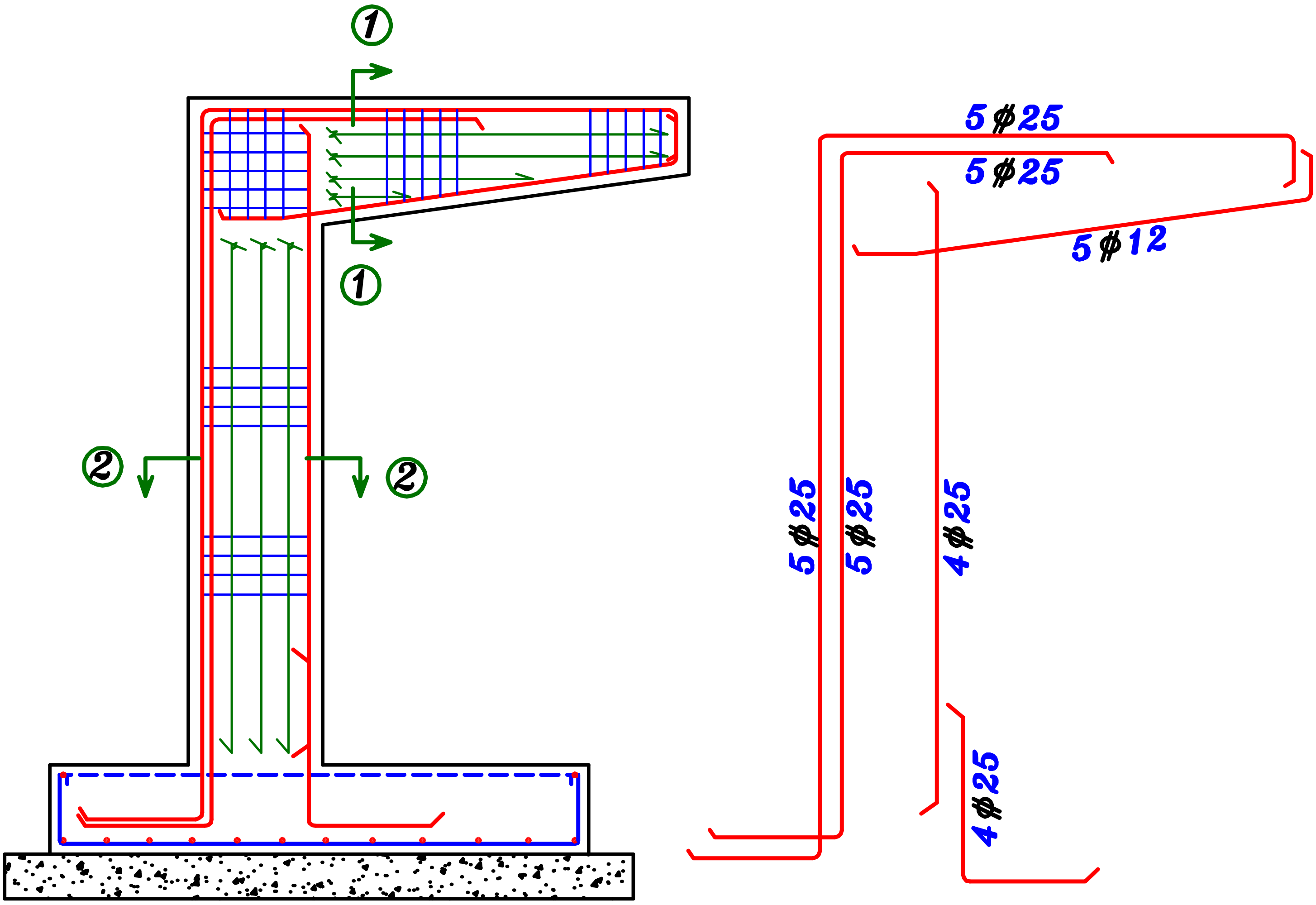
Check $A_{s\,min.}$ $\quad A_{s_{req.}} = 4574 \; mm^2$

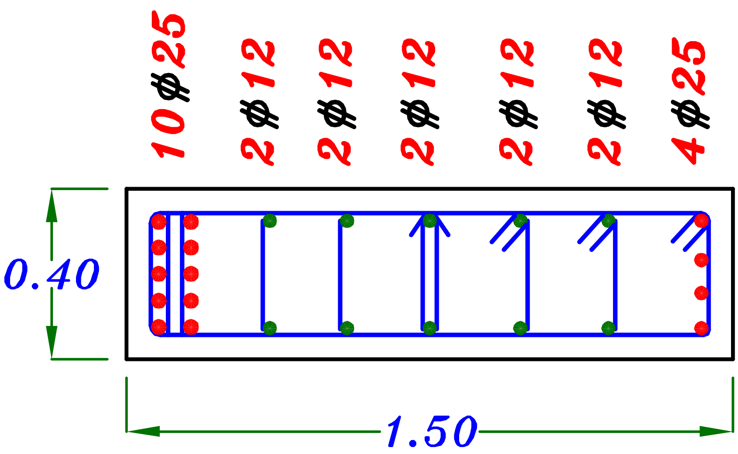
$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1400 = 1750 \; mm^2$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 4574 \; mm^2$ $\quad 10\phi25$

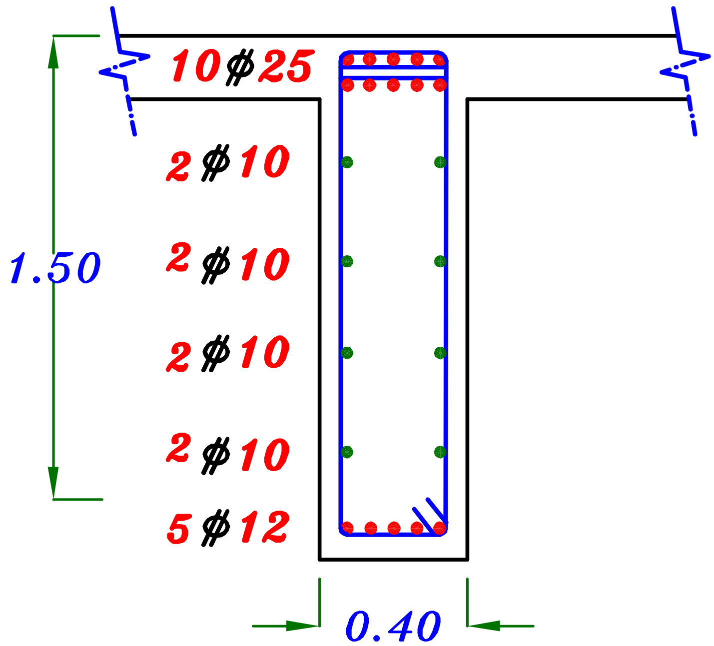
Stirrup Hangers $= 0.4\, A_s = (0.4)\; 4574$ $\quad 4\phi25$

RFT. of the Frame.



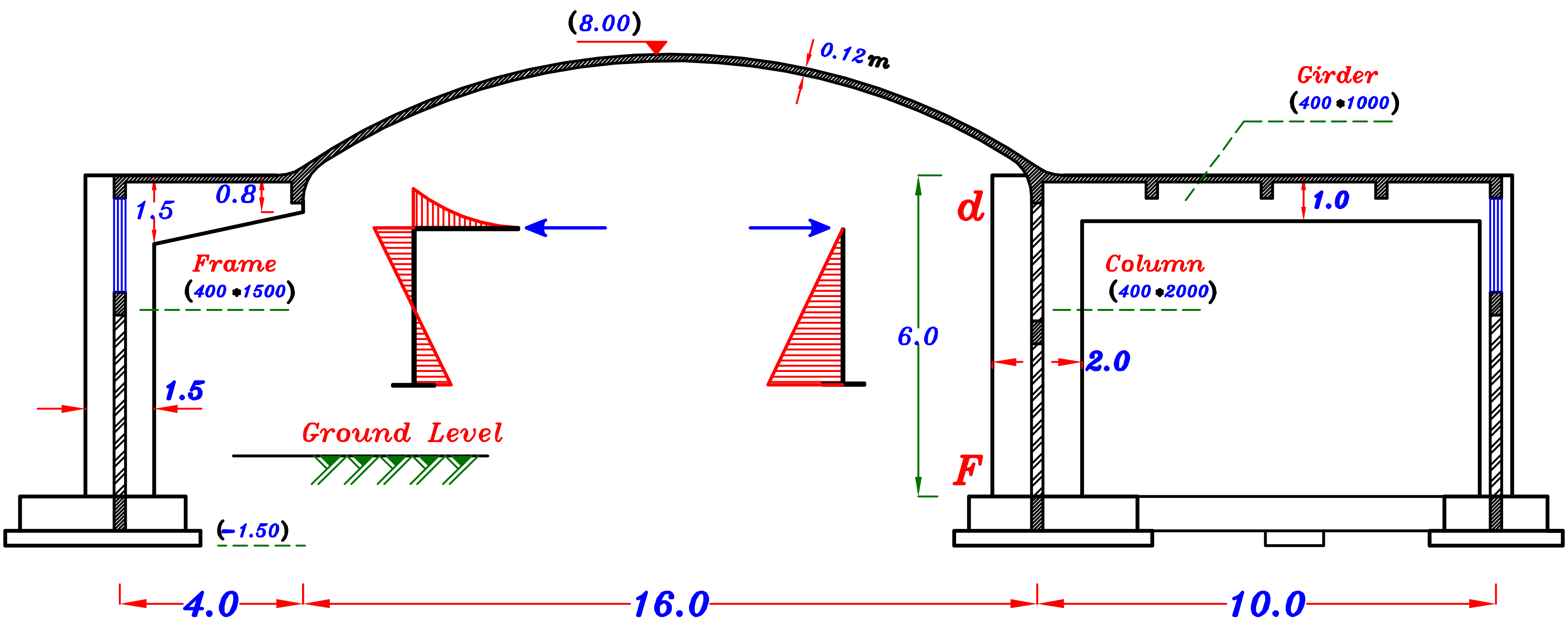


Sec. (2–2)



Sec. (1–1)

④ IF the tie c d is removed.



يتم تكبير ابعاد قطاع العمود d F حتى تزيد **stiffness** العمود
فيسحب كل العزم الناتج عن القوى الافقيه

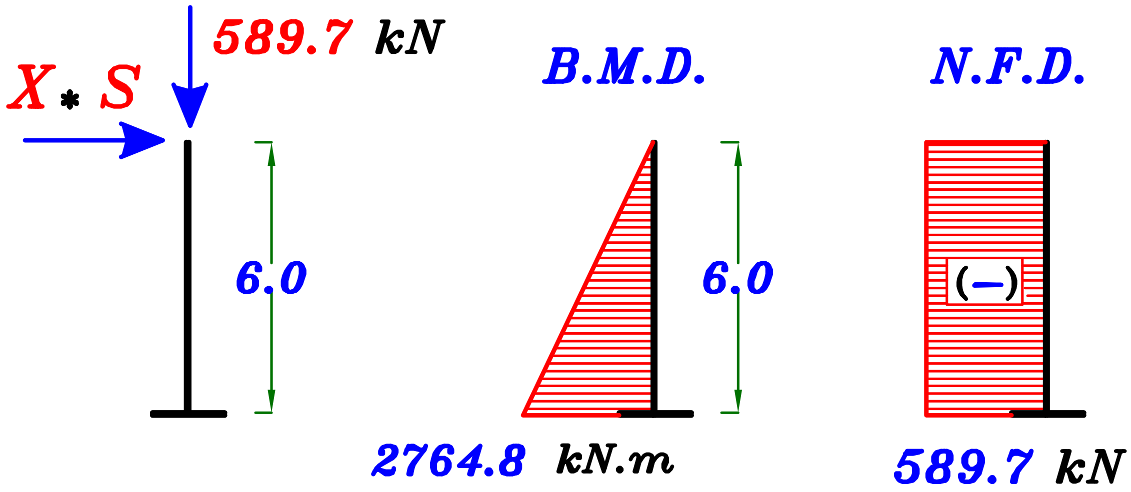
Design the Column Fd

$$P = R_{(Girder)} + R_{(End\ Beam)} = R_{(Girder)} + (o.w. + Y) * S$$

$$P = 276.5 + (4.2 + 48) * 6.0 = 589.7 \text{ kN}$$

$$X * S = 76.8 * 6.0 = 460.8 \text{ kN}$$

$$M = 460.8 * 6.0 = 2764.8 \text{ kN.m}$$



Sec. ① R−sec. Neglect Effect of Buckling.

$M = 2764.8$ kN.m , $P = 589.7$ kN , $b = 400$ mm , $t = 2000$ mm

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{589.7 * 10^3}{25 * 400 * 2000} = 0.029 < 0.04$ (Neglect P)

$$\therefore 1900 = C_1 \sqrt{\frac{2764.8 * 10^6}{25 * 400}} \longrightarrow C_1 = 3.61 \longrightarrow J = 0.788$$

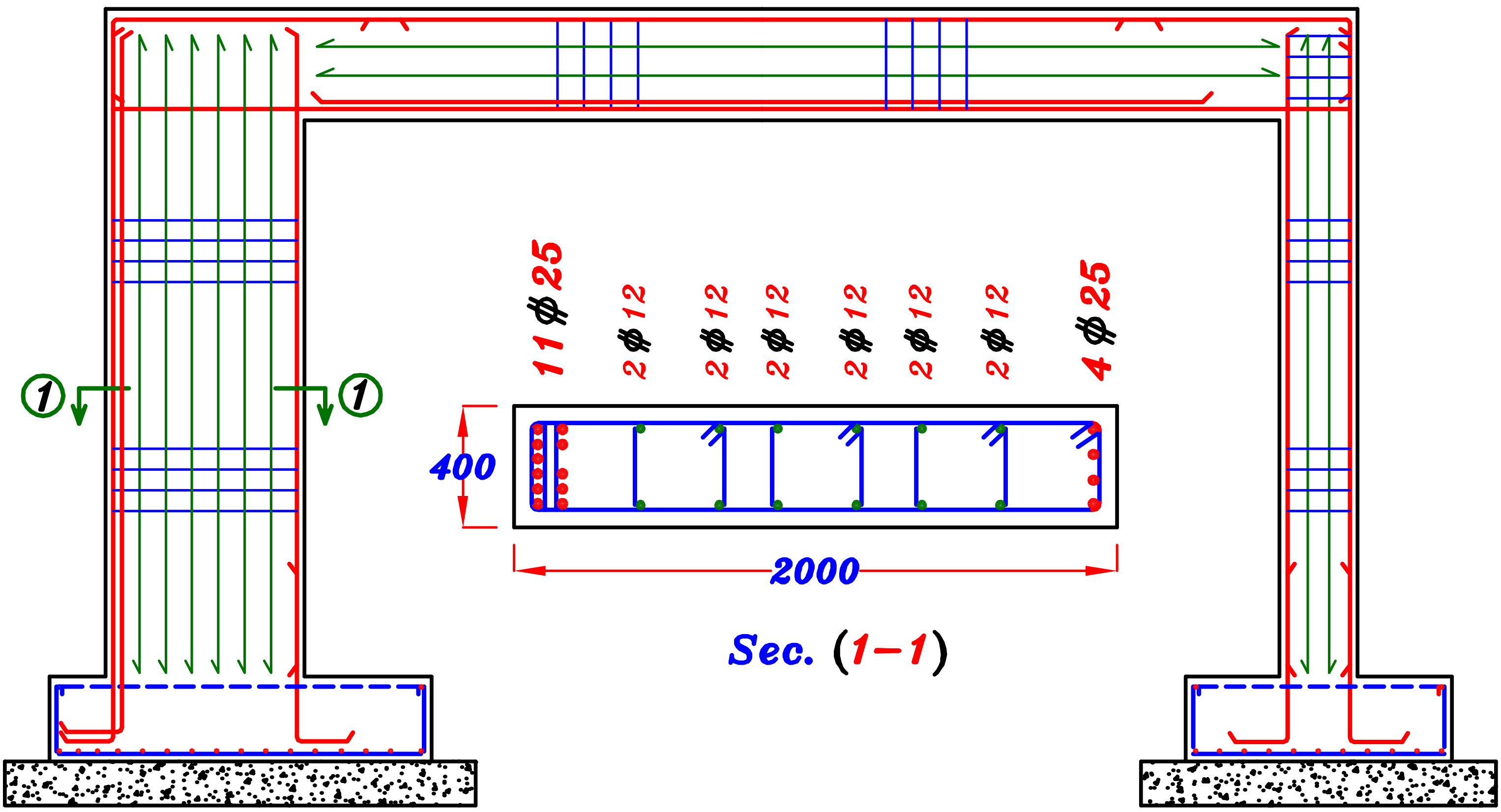
$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{2764.8 * 10^6}{0.788 * 360 * 1900} = 5129.57 \text{ mm}^2$$

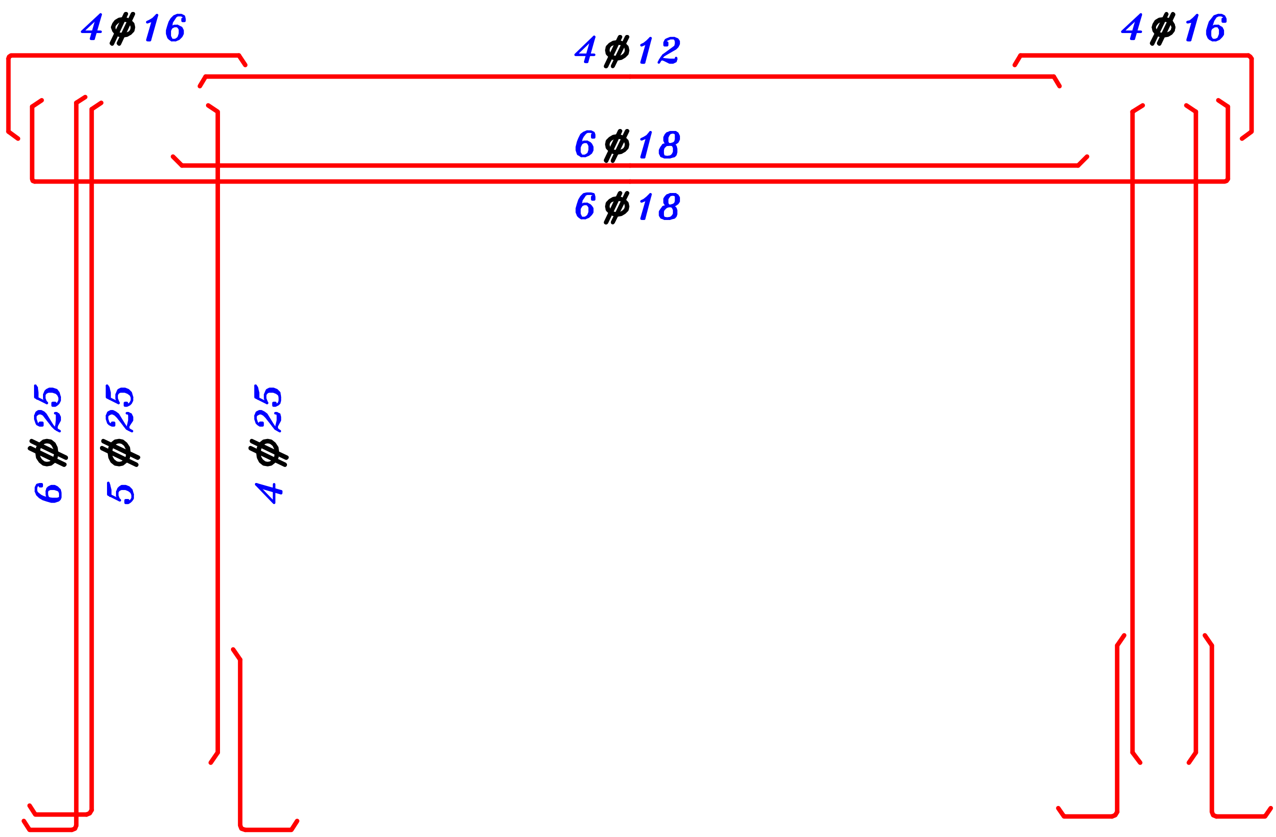
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 5129.57$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1900 = 2375 \text{ mm}^2$$

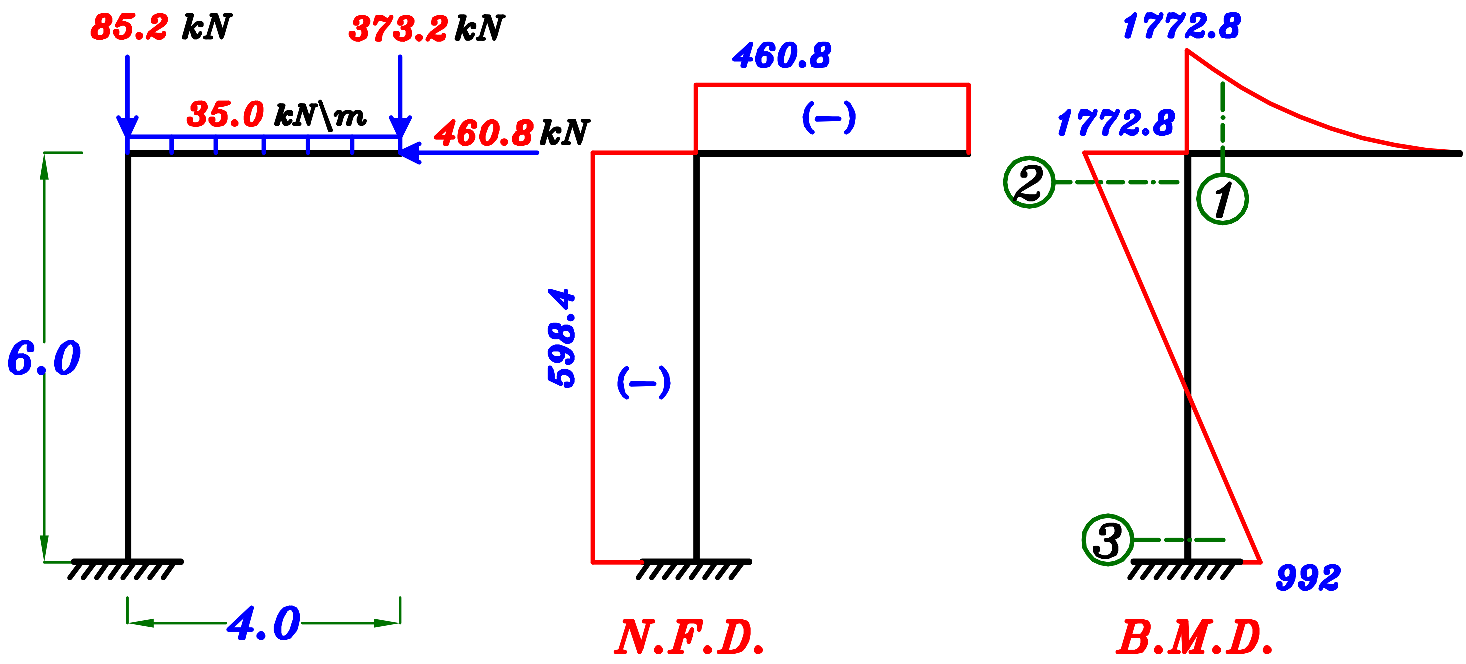
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 5129.57$ mm^2 $\quad 11 \phi 25$

Stirrup Hangers $= 0.4\, A_s = (0.4)\, 5129.57$ $\quad 4 \phi 25$


11 ϕ 25
2 ϕ 12
2 ϕ 12
2 ϕ 12
2 ϕ 12
2 ϕ 12
2 ϕ 12
4 ϕ 25
400
2000
Sec. (1−1)
1
1


4 ϕ 16
4 ϕ 12
4 ϕ 16
6 ϕ 18
6 ϕ 18
6 ϕ 25
5 ϕ 25
4 ϕ 25

2 – Frame a b c. (400 * 1500)



Design of Frame.

Sec. ① & Sec. ②

are the same as before (400 * 1500), A_s = 10 ϕ 25

Sec. ③ R-sec.

$M = 992.0$ kN.m , $P = 598.4$ kN , $b = 400$ mm , $t = 1500$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{598.4 * 10^3}{25 * 400 * 1500} = 0.039 = 0.04$ (we can neglect P)

$\therefore 1400 = C_1 \sqrt{\frac{992.0 * 10^6}{25 * 400}} \longrightarrow C_1 = 4.44 \longrightarrow J = 0.815$

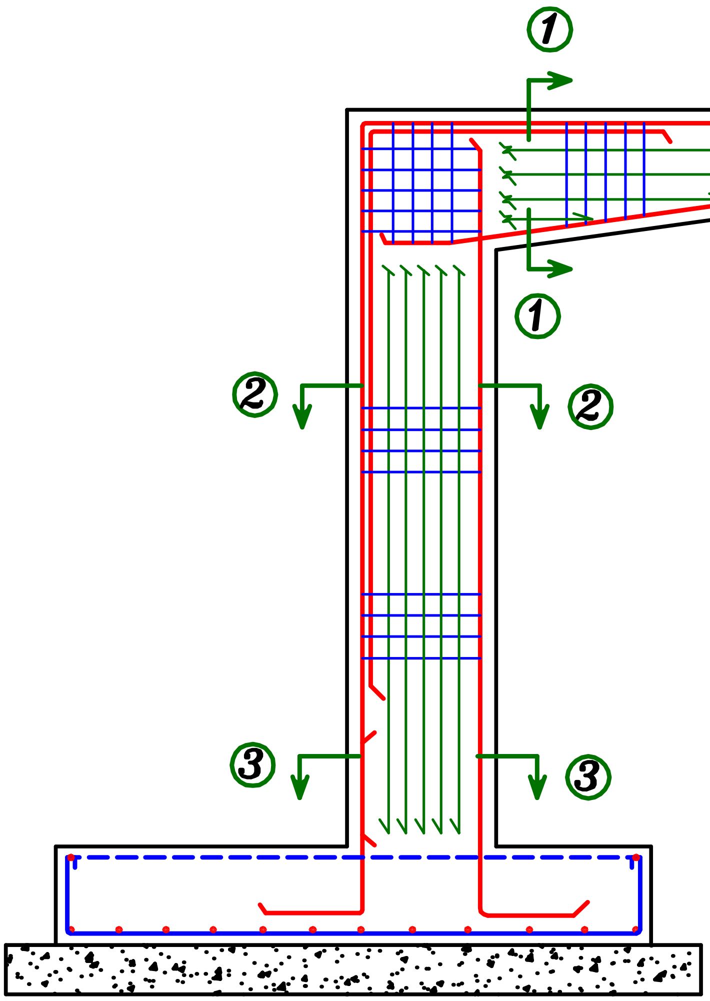
$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{992.0 * 10^6}{0.815 * 360 * 1400} = 2415$ mm^2

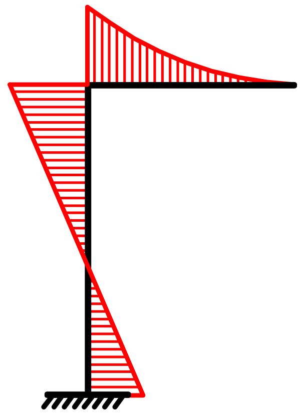
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 2415$ mm^2

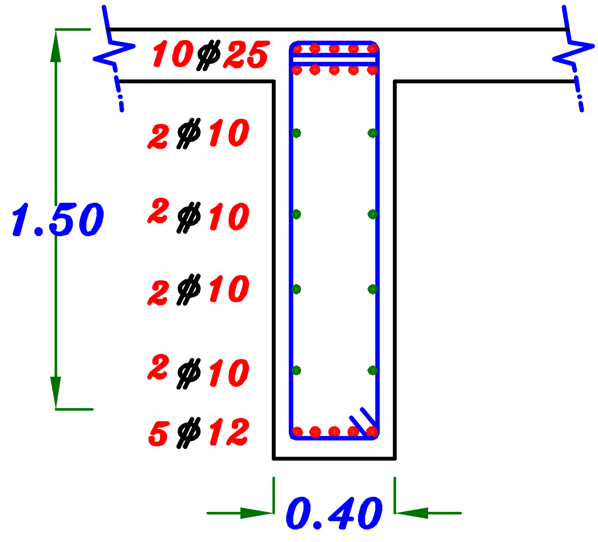
$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1400 = 1750$ mm^2

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2415$ mm^2 5 ϕ 25

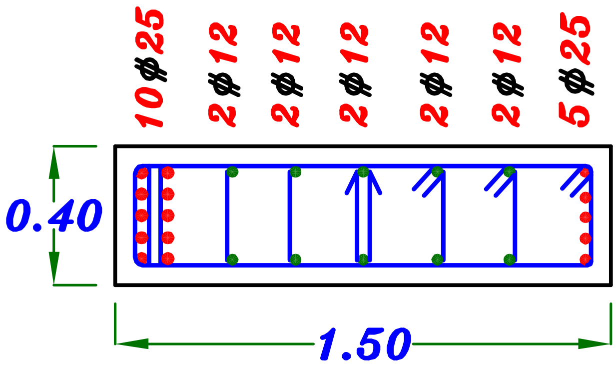
RFT. of the Frame.



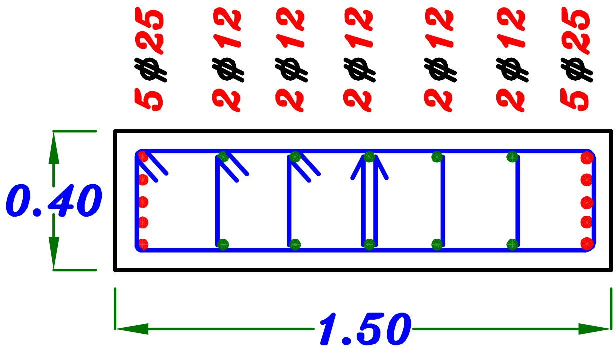




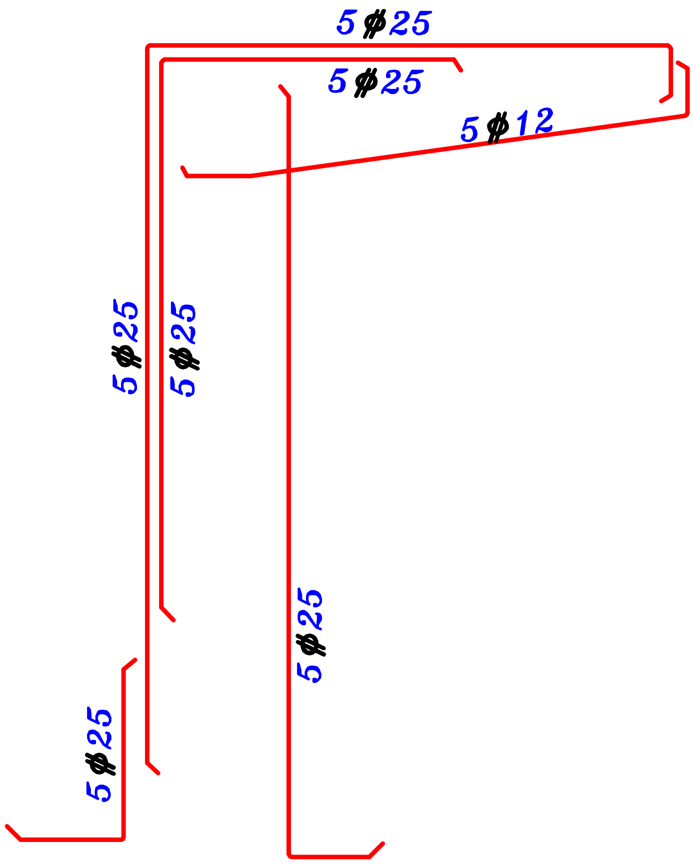
Sec. (1−1)



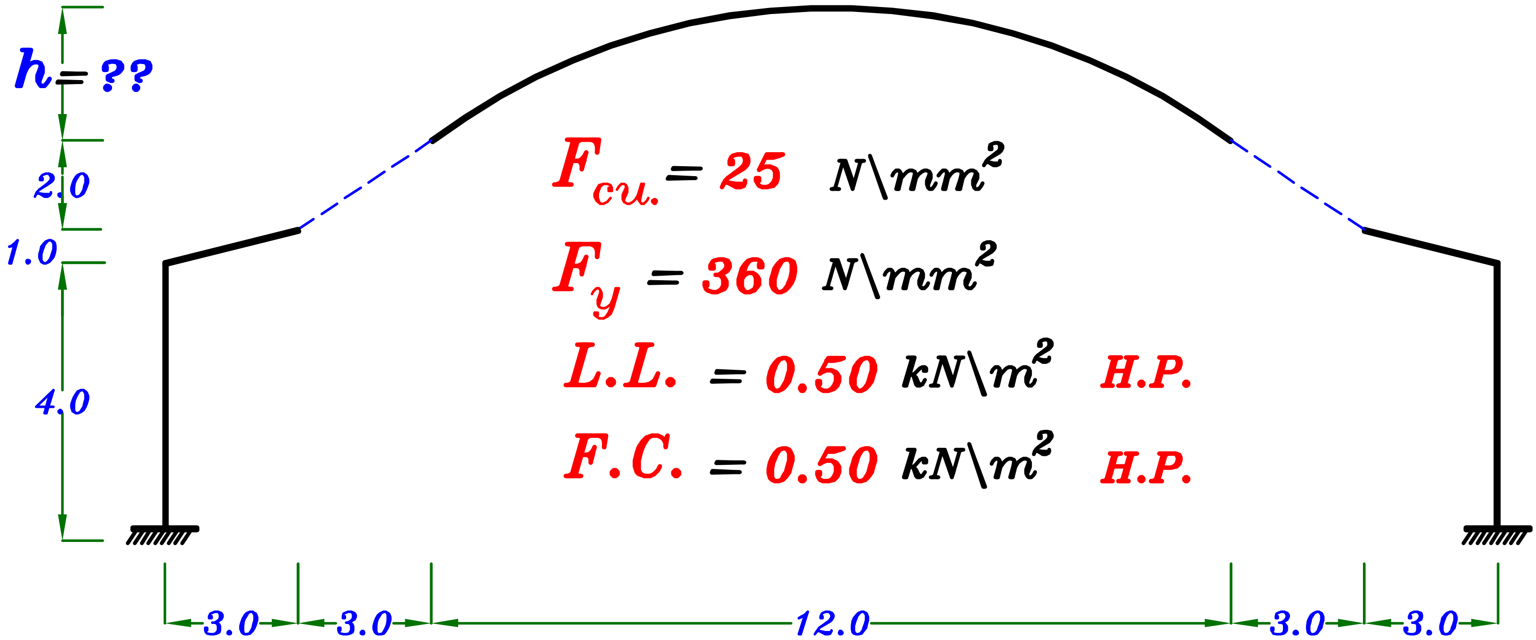
Sec. (2−2)



Sec. (3−3)



Example.



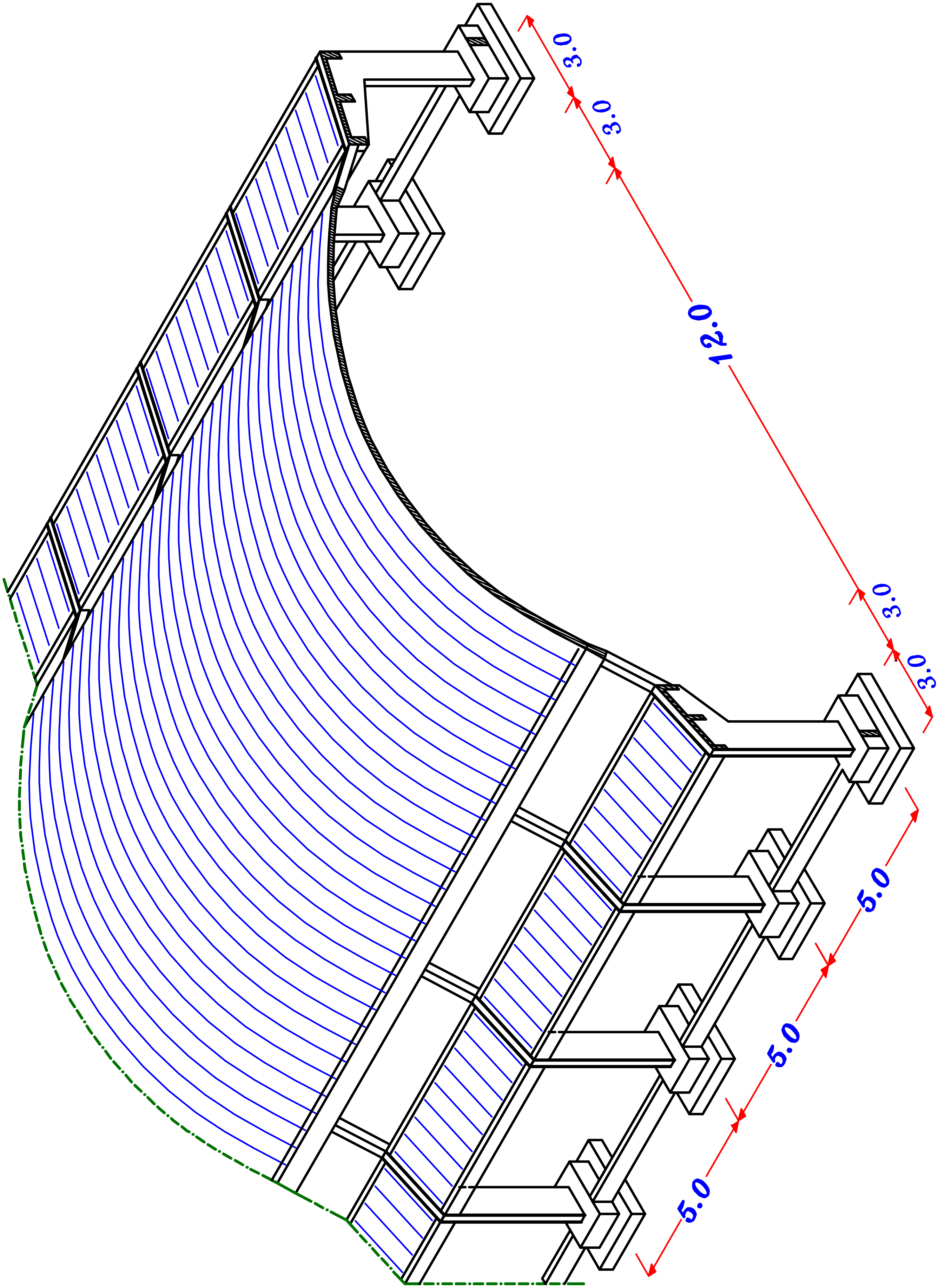
The Fig. shows a sectional elevation of covering area (24.0 m * 40.0 m).

It is Required to :

1. ***Suggest a statical system to cover the area and suggest value For h = ??
and draw a sectional elevation to scale 1:50
showing the dimensions of all concrete elements.***

2. ***Design the main supporting elements.
and draw details of RFT. in elevation to scale 1:50***

3. ***IF the windows is vertical (the span of arch slab = 18.0 m)
make a necessary modifications to the suggested system, once with a Tie and once without Tie.
and draw a sectional elevation to scale 1:50
For the two modified systems.***

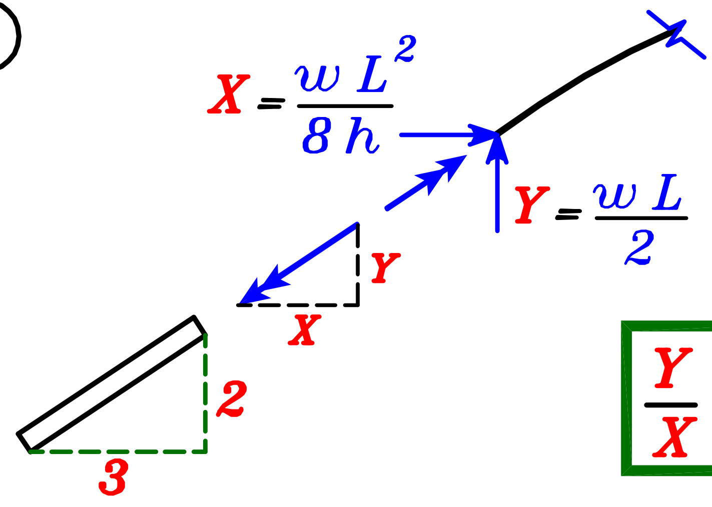

3.0
3.0
12.0
3.0
3.0
5.0
5.0
5.0

يجب أن تكون محصلة القوى عمودية على ال **Post**

حتى لا تعمل على ال **Post** على ال **B.M.**

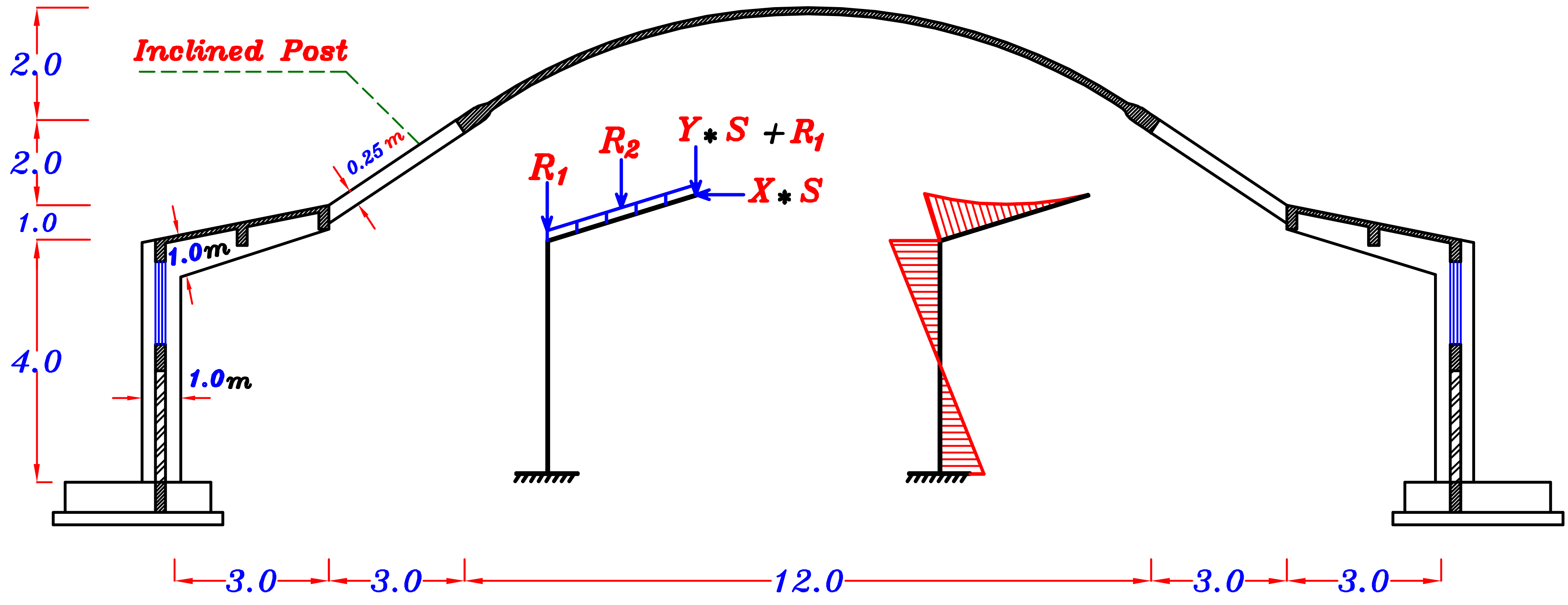
حيث أنة مصمم على **N.F.** فقط

①



$$X = \frac{w\,L^2}{8\,h} \qquad Y = \frac{w\,L}{2}$$

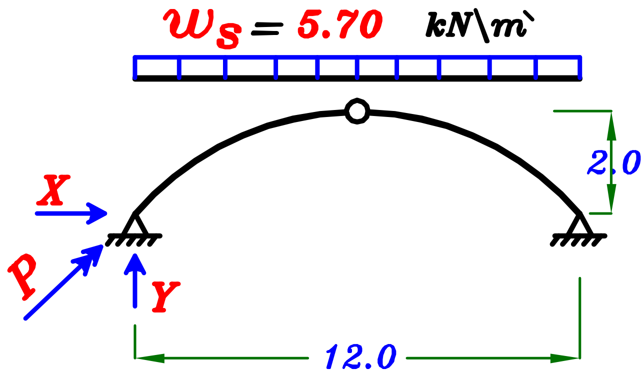
$$\boxed{\frac{Y}{X} = \frac{2}{3}} \longrightarrow \frac{wL \backslash 2}{wL^2 \backslash 8\,h} = \frac{2}{3} \longrightarrow \boxed{h = 2.0\ m}$$



Design the Arch Slab.

Take $t_s = 120\ mm$

$$(w_s)_{U.L.} = 1.4\ (t_s\ \gamma_c + F.C.) + 1.6\ (L.L.)$$



$$(w_s)_{U.L.} = 1.4\,(0.12 * 25 + 0.50) + 1.6\,(0.50)$$

$$= 5.70\ \ kN\backslash m^2\ \ (H.P.)$$

To Get N.F.

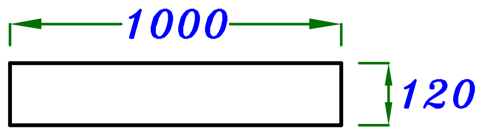
$$Y = \frac{w\ L}{2} = \frac{5.70 * 12}{2} = 34.2\ \ kN\backslash m$$

$$X = \frac{w\ L^2}{8\ h} = \frac{5.70 * 12^2}{8 * 2.0} = 51.3\ \ kN\backslash m$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{34.2^2 + 51.3^2} = 61.65\ kN$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



$\therefore$ Designed as a Column.

$$\therefore\ P_{U.L.} = 0.35\ A_c\ F_{cu} + 0.67\ A_s\ F_y$$

Take $A_c = 120 * 1000 = 120000\ mm^2$

$$\therefore\ 61.65 * 10^3 = 0.35\,(120000)(25) + 0.67\ A_s\ (360)$$

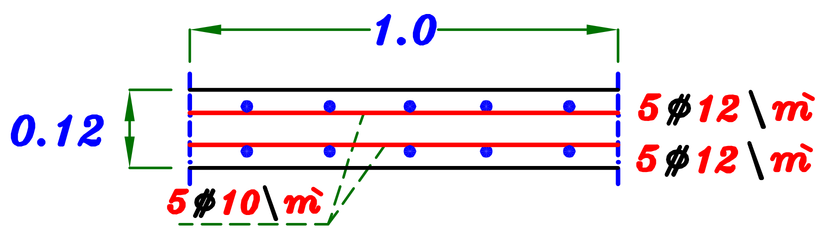
$$\therefore\ A_s = -\ 4097\ \ mm^2 = -\ (Ve)\ \text{Value}$$

$$\therefore\ \text{Take}\ \ A_s = A_{s_{min.}} = \frac{0.8}{100} * b * t$$

$$\therefore\ A_s = \frac{0.8}{100} * 120 * 1000 = 960\ \ mm^2 = A_{s\ total}$$

$$\therefore\ \text{Upper Steel \& Lower Steel} = \frac{A_{s\ total}}{2} = \frac{960}{2} = 480\ mm^2$$

5ϕ12\m`



Design the End Beams.

$\theta = 33.70^\circ$

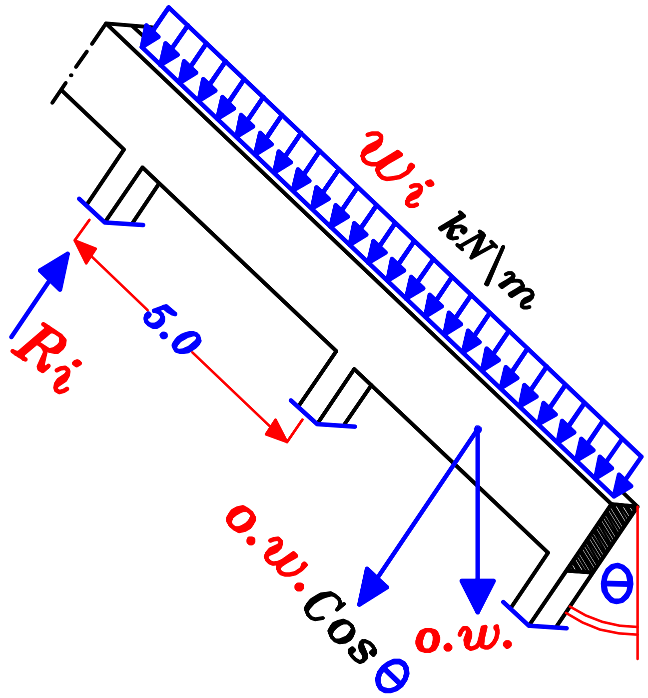
Take $o.w. = 7.0$ kN

$P = \sqrt{34.2^2 + 51.3^2} = 61.65$ kN

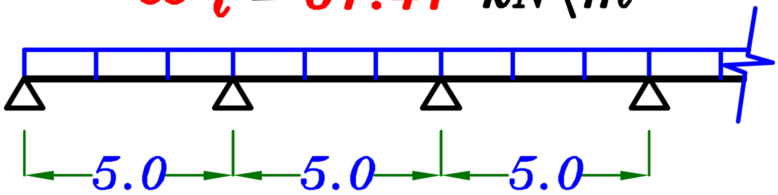
$w_i = P + o.w. * \cos\theta$

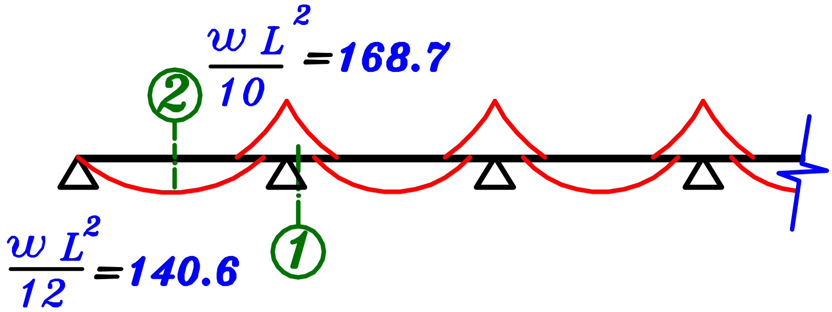
$w_i = 61.65 + 7.0 * \cos 33.70$

$= 67.47$ kN/m



$w_i = 67.47$ kN\m





Sec. 1

$M_{U.L.} = 168.7$ kN.m R-Sec.

- Take $C_1 = 3.50 \longrightarrow J = 0.78$

- Get $d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\dfrac{168.7 * 10^6}{25 * 250}} = 575.0$ mm

- Take $\boxed{d = 600 \text{ mm}}$, $\boxed{t = 650 \text{ mm}}$

- Get $A_s = \dfrac{M_{U.L.}}{J\, F_y\, d} = \dfrac{168.7 * 10^6}{0.78 * 360 * 575.0} = 1044.0 \text{ mm}^2$

Check $A_{s\,min.}$ $A_{s_{req.}} = 1044.0\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 600 = 468.7\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1044.0\ mm^2$ 5 ϕ 18

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \text{ bars}$$

Sec. ②

$M_{U.L.} = 140.6\ kN.m$

$$600 = C_1 \sqrt{\frac{140.6 * 10^6}{25 * 250}} \longrightarrow C_1 = 4.0 \longrightarrow J = 0.803$$

$$A_s = \frac{140.6 * 10^6}{0.803 * 360 * 600} = 810.6\ mm^2$$

Check $A_{s\,min.}$ $A_{s_{req.}} = 810.6\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 600 = 468.7\ mm^2$$

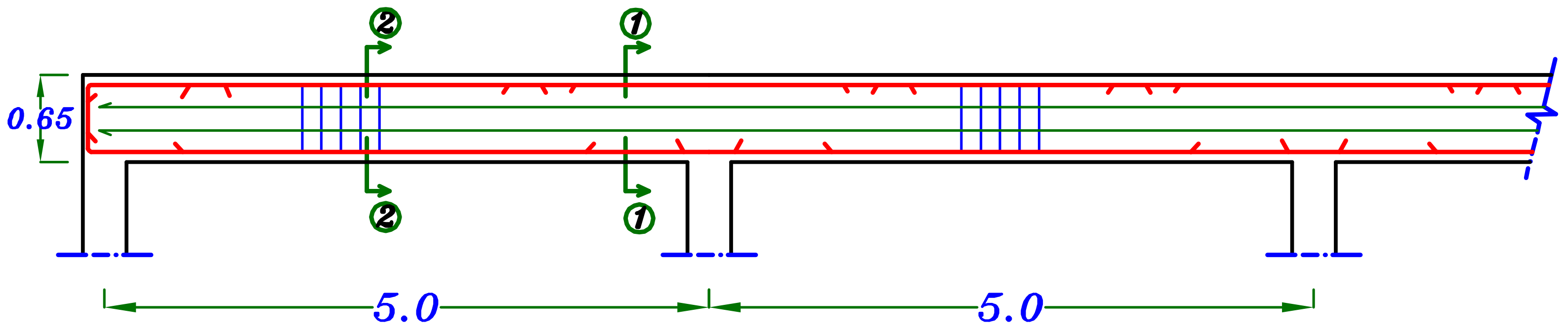
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 810.6\ mm^2$ 4 ϕ 18

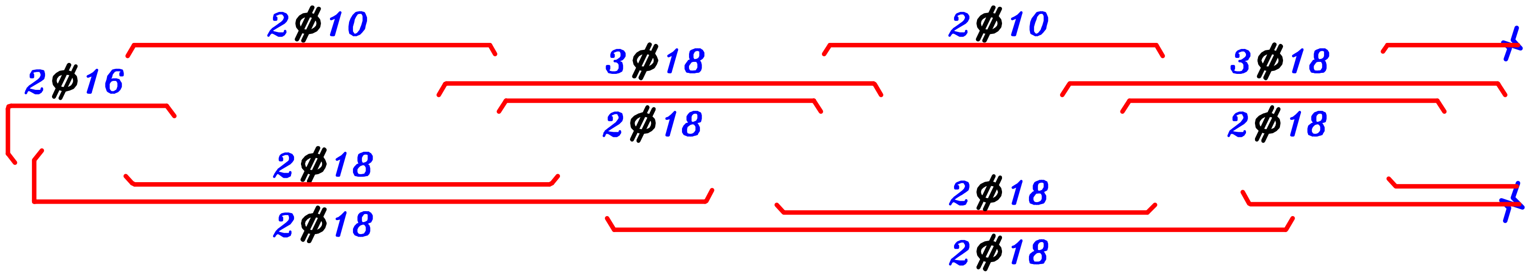
Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 810.6$ 2 ϕ 10

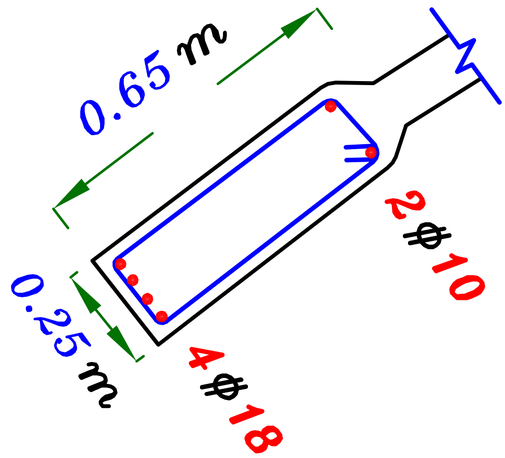
Reaction of inclined Beam.

$$R_i = w_i * S = 67.47 * 5.0 = 337.35\ kN$$

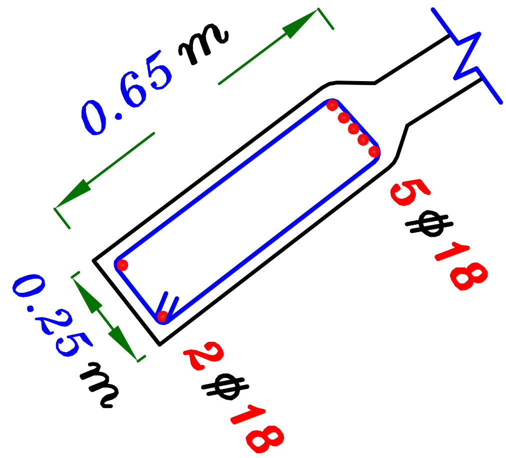
RFT. of Inclined End Beam.





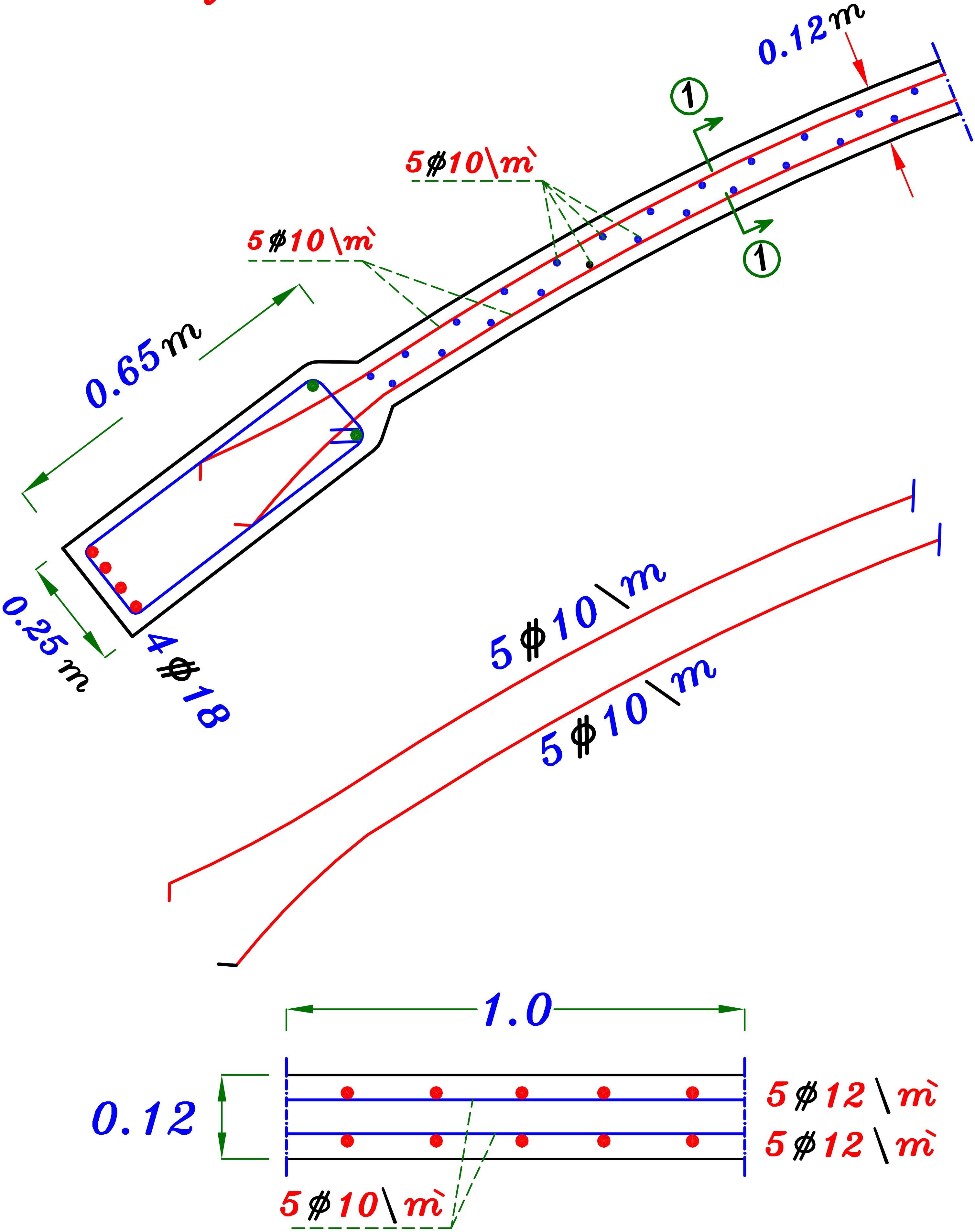


Sec. (2–2)



Sec. (1–1)

RFT. of the Arch slab.

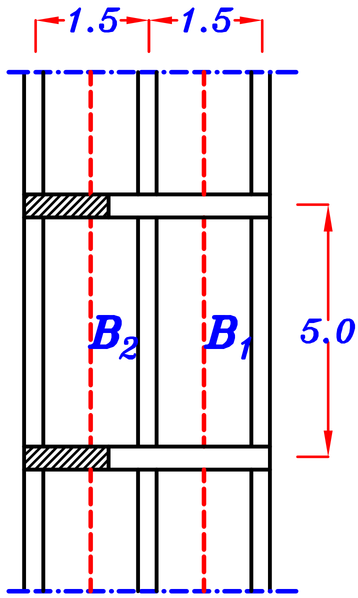


Sec. (1–1)

Loads on the Frame.

$$(w_s)_{U.L.} = 1.4\,(0.12 * 25 + 0.50) + 1.6\,(0.50) = 5.70 \ kN\backslash m^2$$

Take $(o.w.)_{Beams} = 1.4 * 3.0 = 4.20 \ kN\backslash m`$ U.L.



B_1

$$w_a = o.w. + w_s \frac{L_s}{2} = 4.20 + (5.70)\left(\frac{1.5}{2}\right) = 8.475 \ kN\backslash m`$$

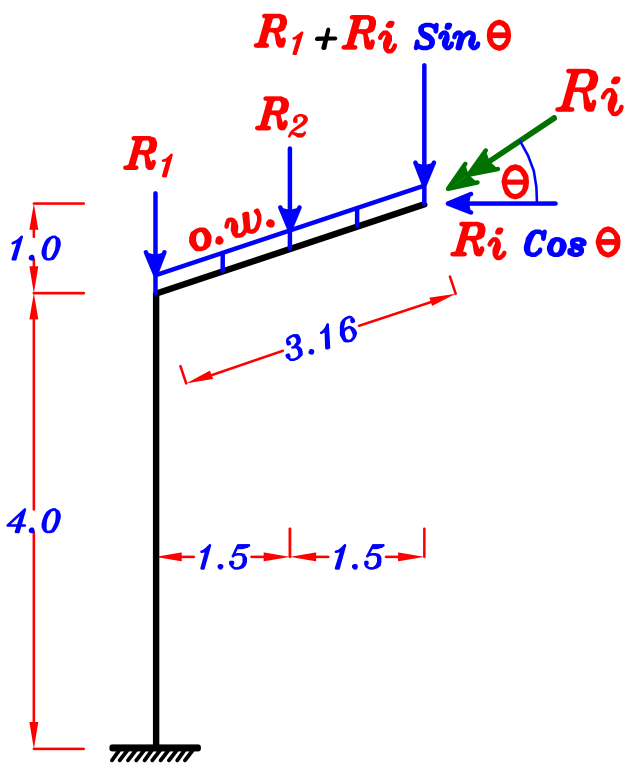
$$R_1 = 8.475 * 5.0 = 42.375 \ kN \qquad \boxed{R_1 = 42.375 \ kN}$$

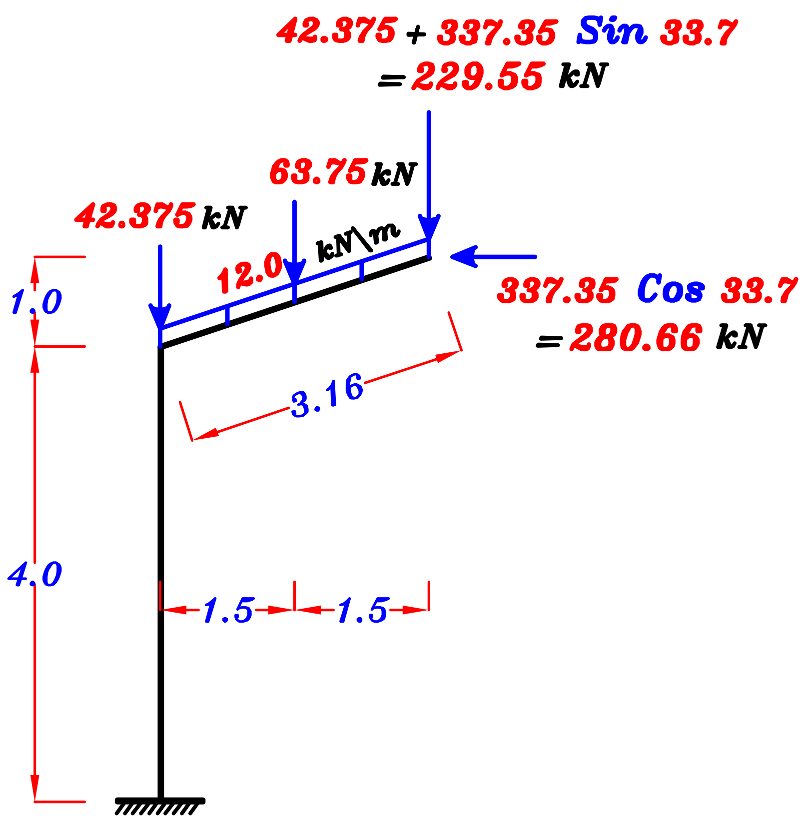
B_2

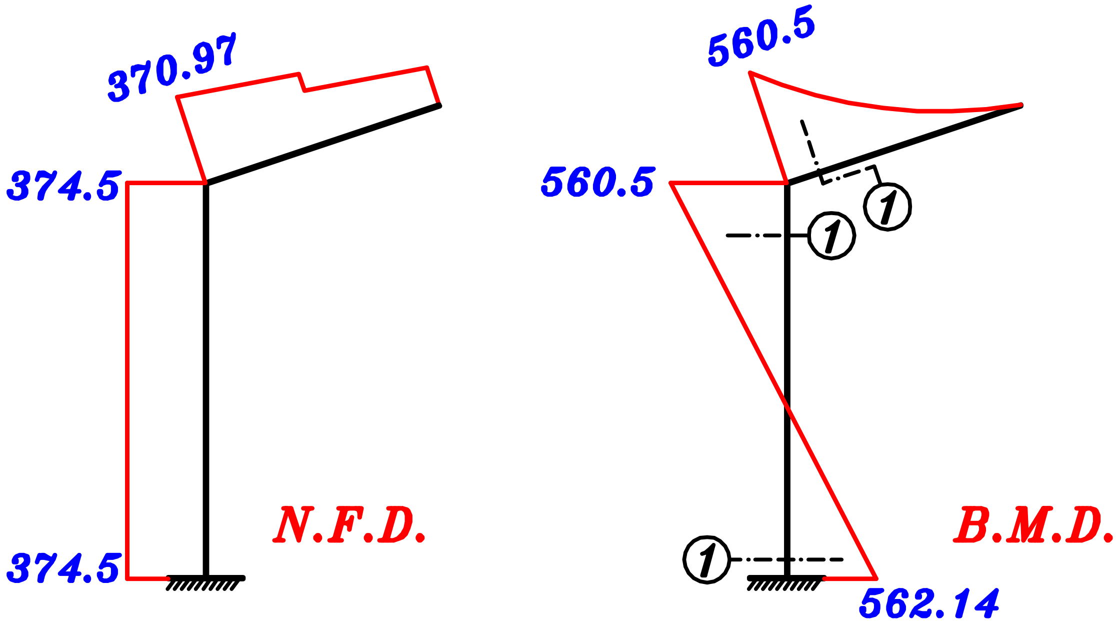
$$w_a = o.w. + 2\,w_s \frac{L_s}{2} = 4.20 + 2\,(5.70)\left(\frac{1.5}{2}\right) = 12.75 \ kN\backslash m`$$

$$R_2 = 12.75 * 5.0 = 63.75 \ kN \qquad \boxed{R_2 = 63.75 \ kN}$$

Take $(o.w.)_{Frame} = 12.0 \ kN\backslash m`$ U.L.







Sec. 1 R-sec. Neglect Effect of Buckling.

$M = 562.14$ kN.m , $P = 374.5$ kN , $b = 350$ mm, $t = 1000$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{374.5 * 10^3}{25 * 350 * 1000} = 0.0428 > 0.04$ (Don't neglect P)

$e = \frac{M}{P} = \frac{562.14}{374.5} = 1.50$ m $\quad \therefore \frac{e}{t} = \frac{1.50}{1.0} = 1.50$ m $> 0.5 \xrightarrow{use} e_s$

$e_s = e + \frac{t}{2} - c = 1.50 + \frac{1.0}{2} - 0.05 = 1.95$ m

$M_s = P * e_s = 374.5 * 1.95 = 730.27$ kN.m

$\therefore \; 950 = C_1 \sqrt{\frac{730.27 * 10^6}{25 * 350}} \longrightarrow C_1 = 3.29 \longrightarrow J = 0.767$

$\therefore A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)} = \frac{730.27 * 10^6}{0.767 * 360 * 950} - \frac{374.5 * 10^3}{(360 \backslash 1.15)}$

$= 1587.6$ mm^2

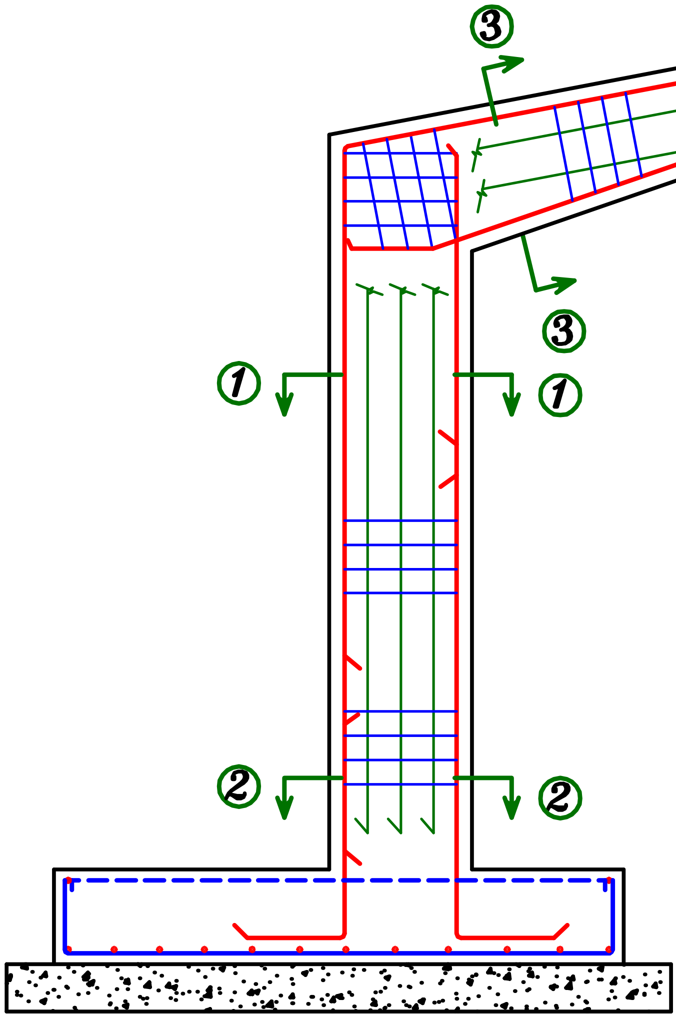
Check $A_{s\,min.}$ $\quad A_{s_{req.}} = 1587.6$ mm^2

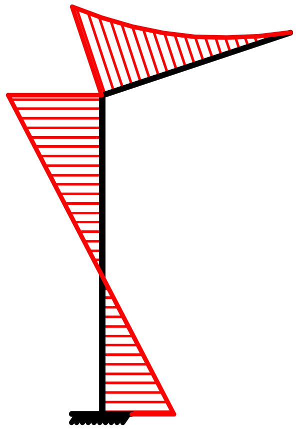
$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 950 = 1039$ mm^2

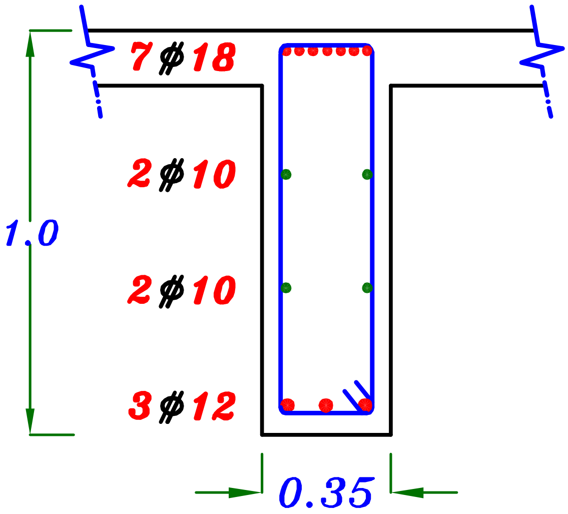
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1587.6$ mm^2 $\quad 7 \phi 18$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 1587.6 \quad 3 \phi 12$

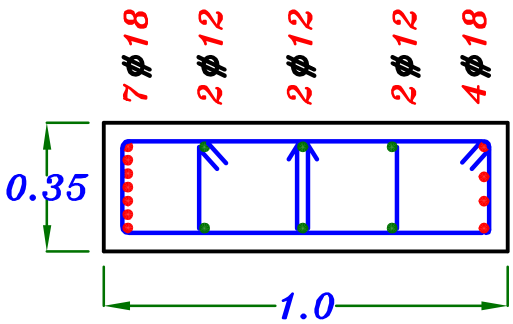
RFT. of the Frame.



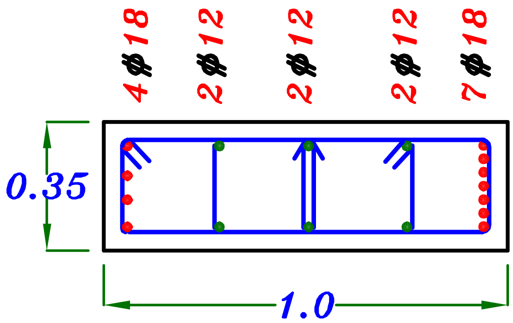




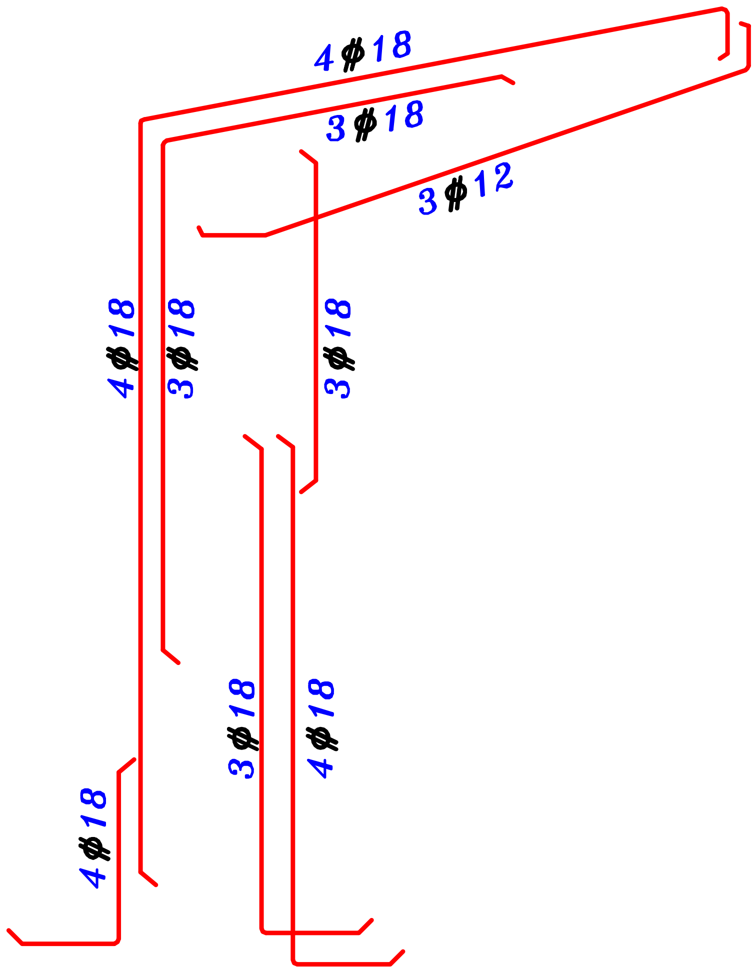
Sec. (3–3)



Sec. (1–1)



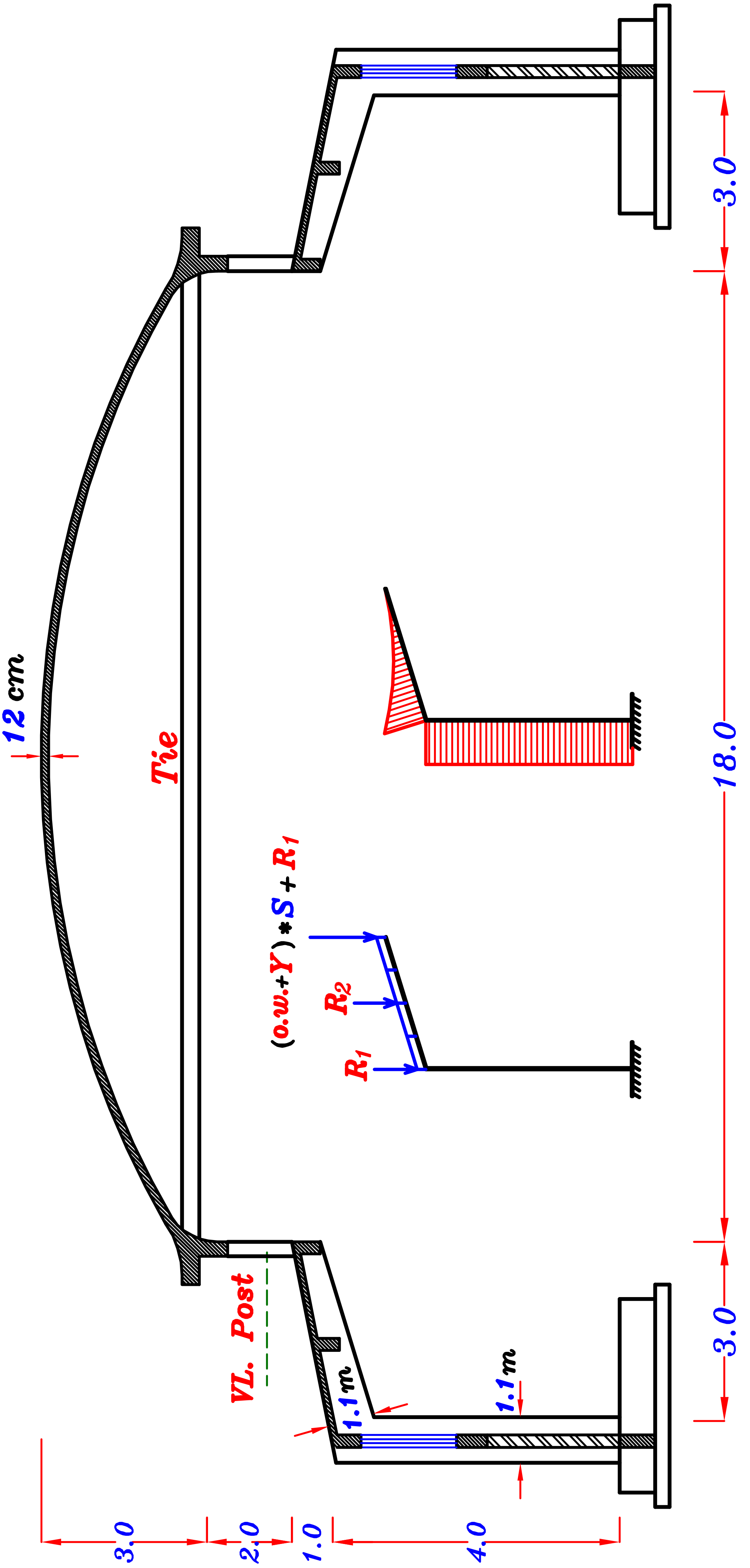
Sec. (2–2)

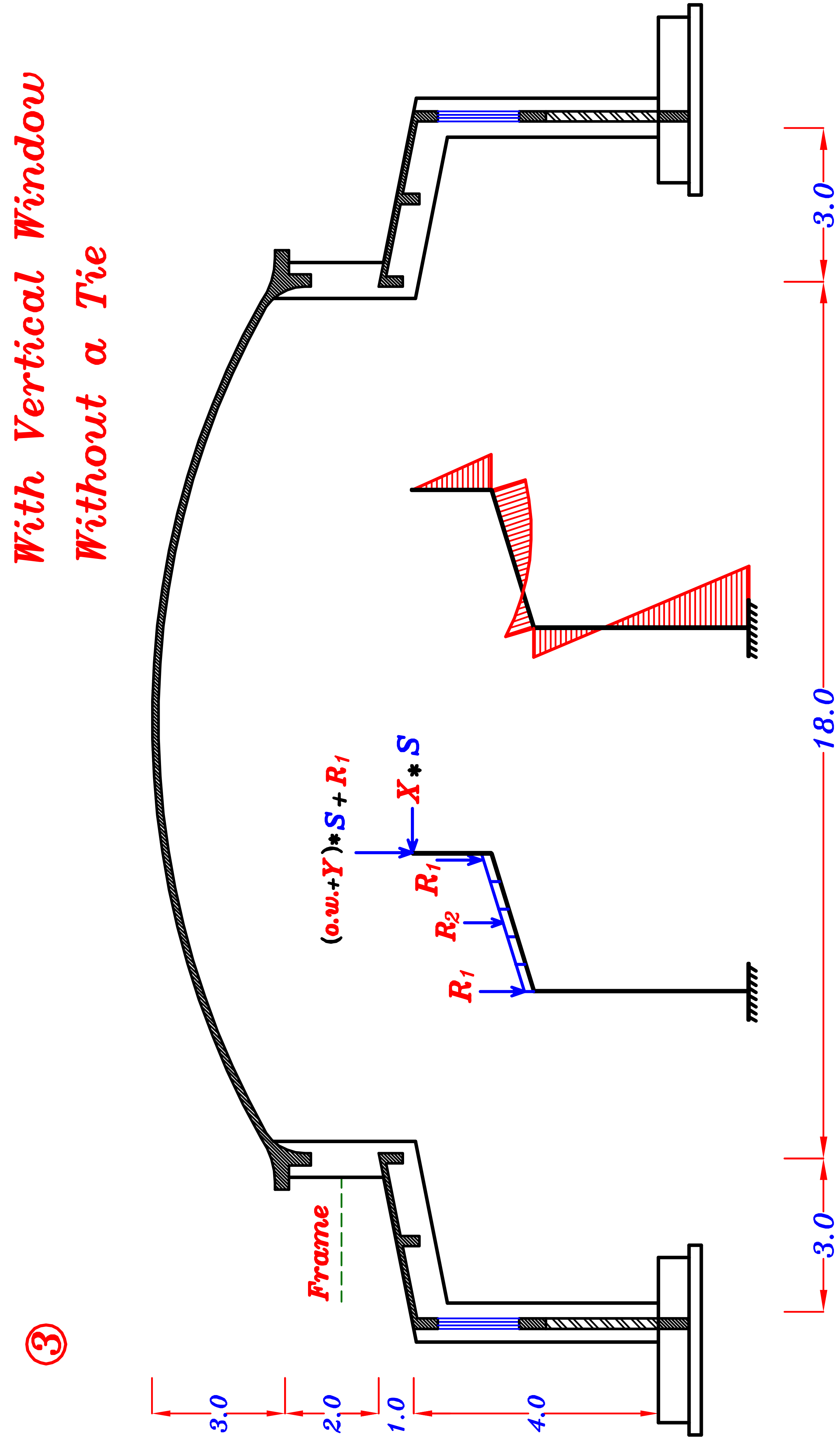


③

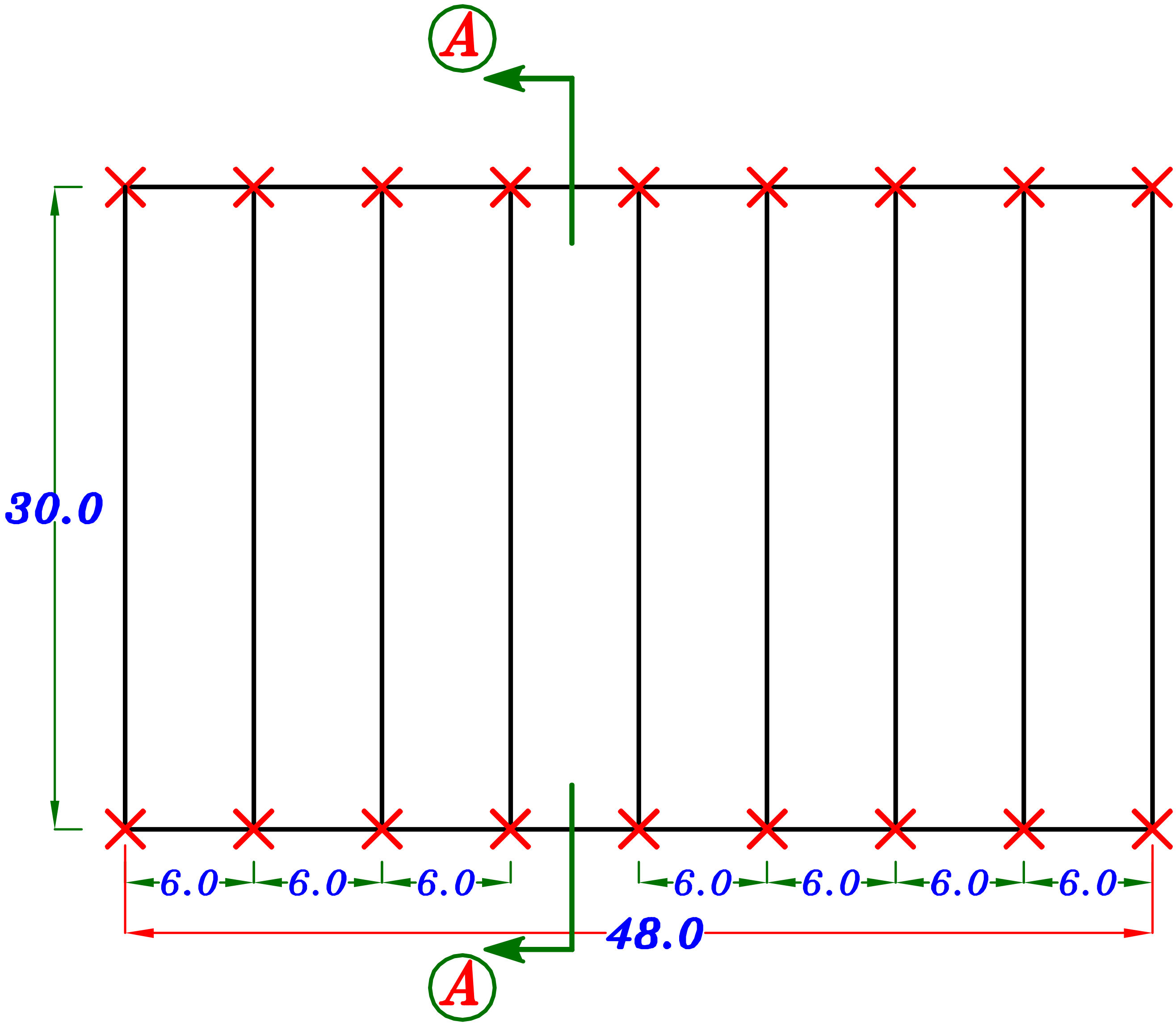
With Vertical Window

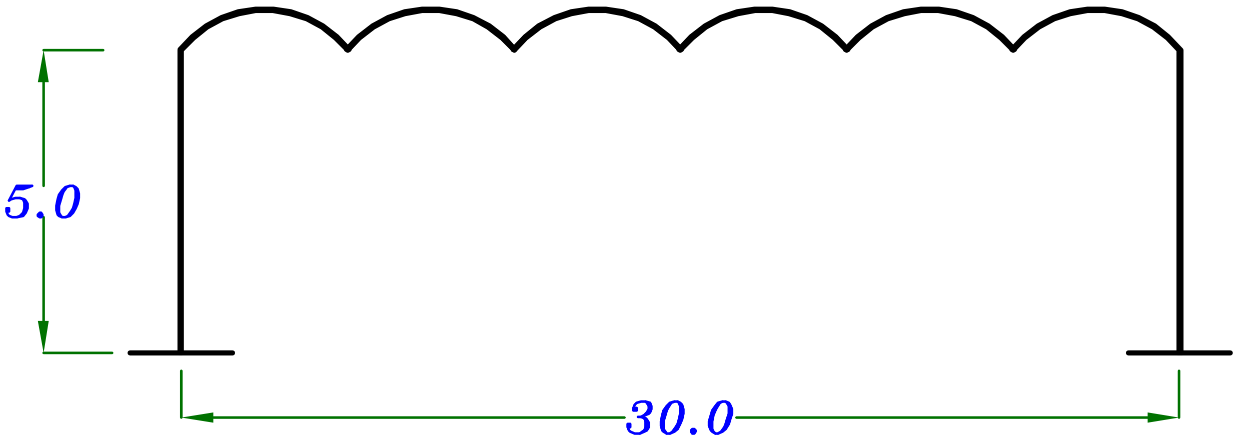
With a Tie



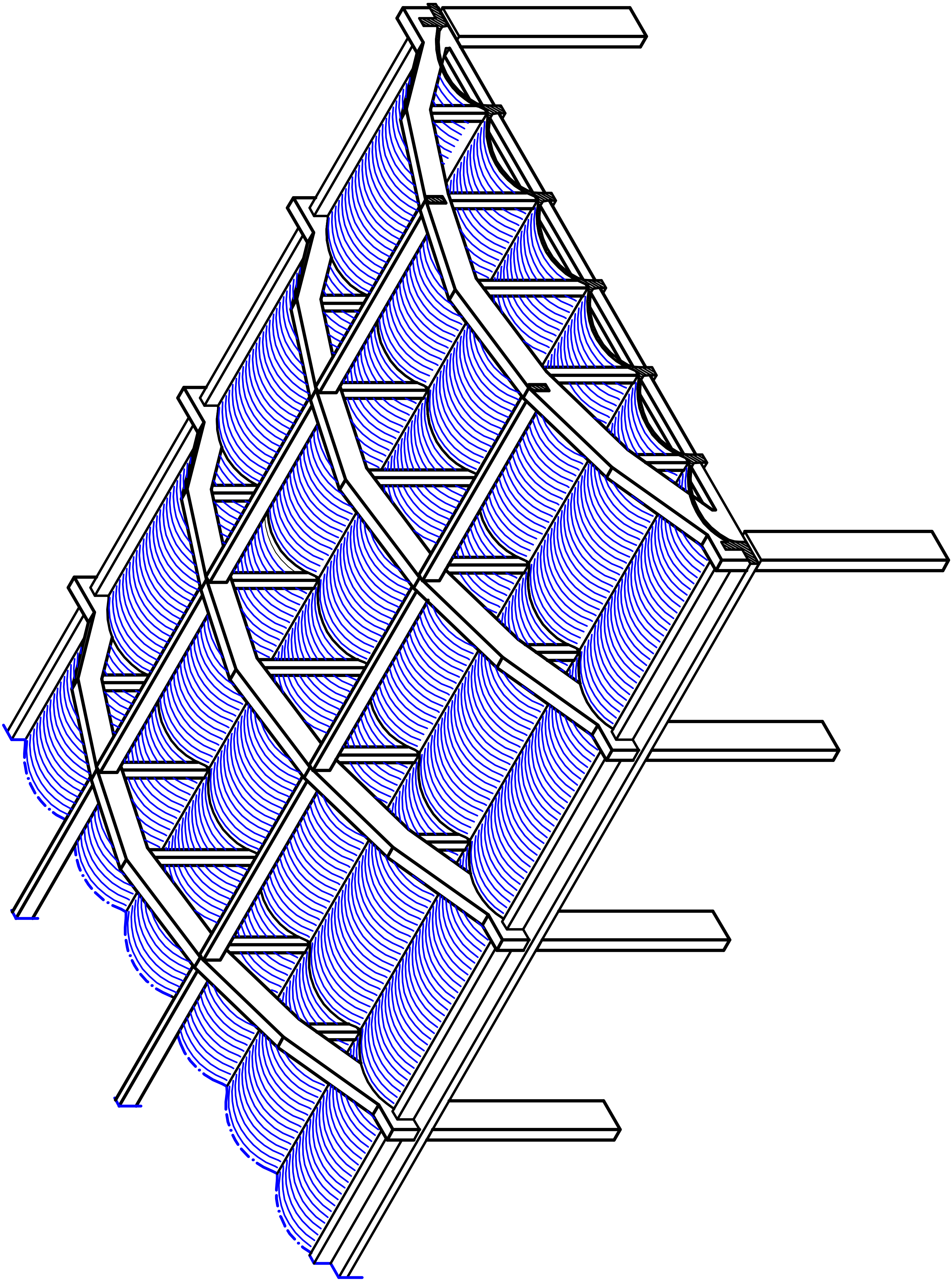

With Vertical Window
Without a Tie
3
Frame
(o.w.+Y)*S+R1
X*S
R1
R2
R1
3.0
2.0
1.0
4.0
3.0
18.0
3.0

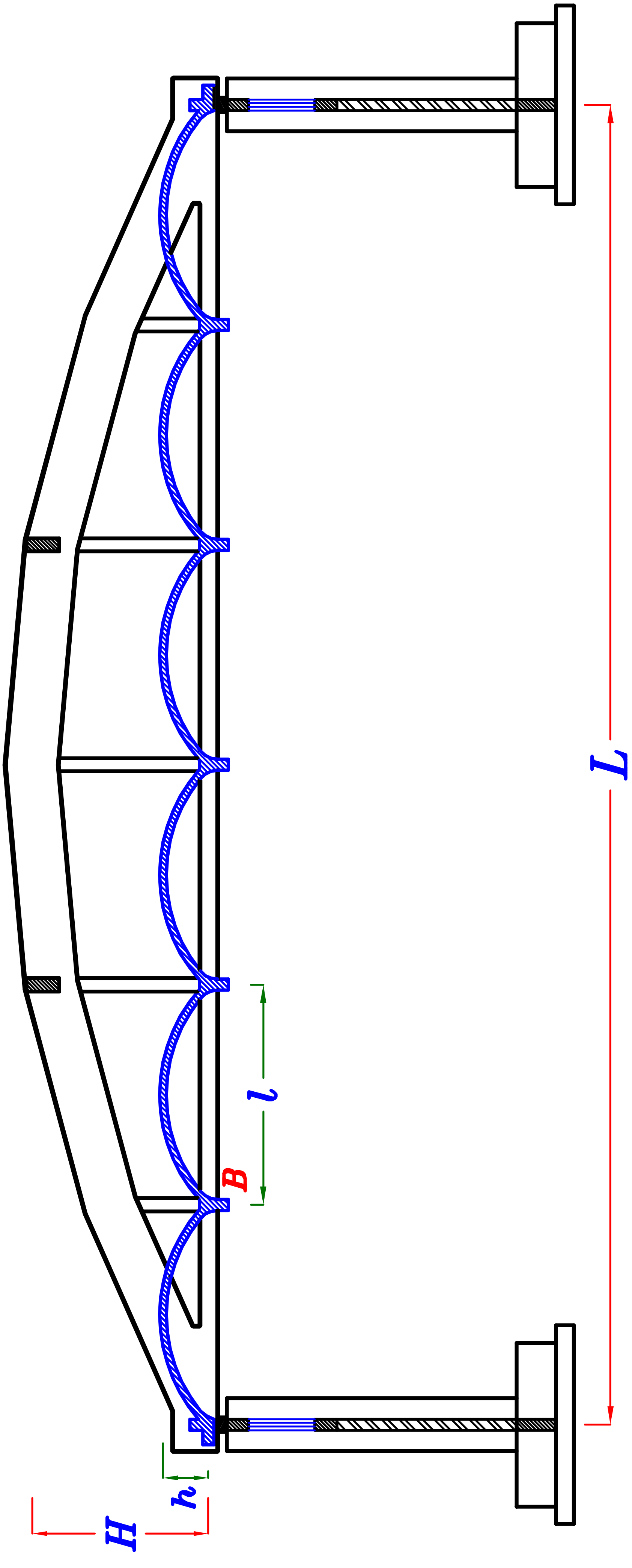
Example.



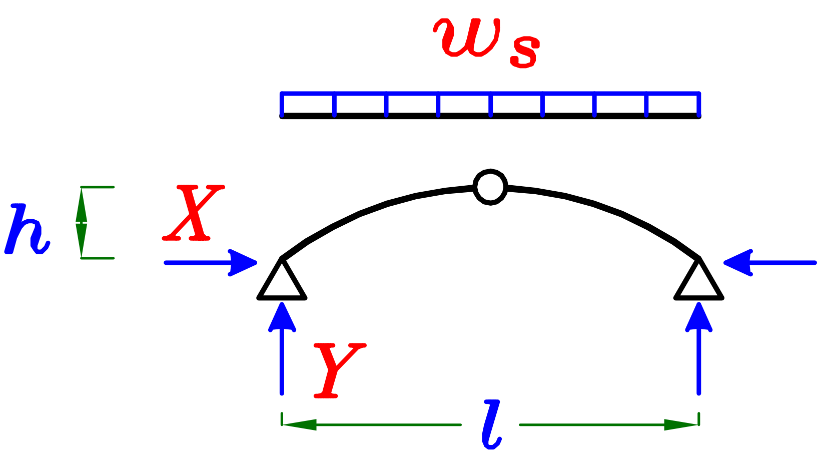


Sec. (A–A)




L
l
B
h
H

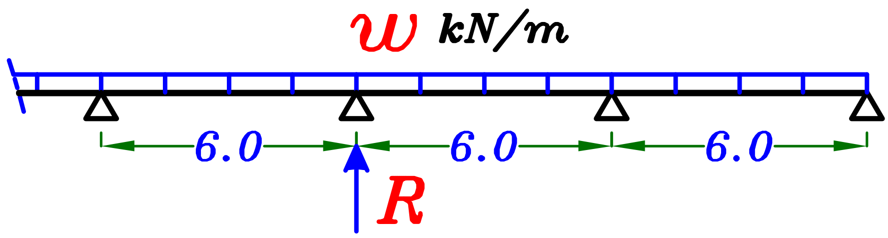
Arch slab



$$Y = \frac{w_s\, l}{2}$$

$$X = \frac{w_s\, l^2}{8\, h}$$

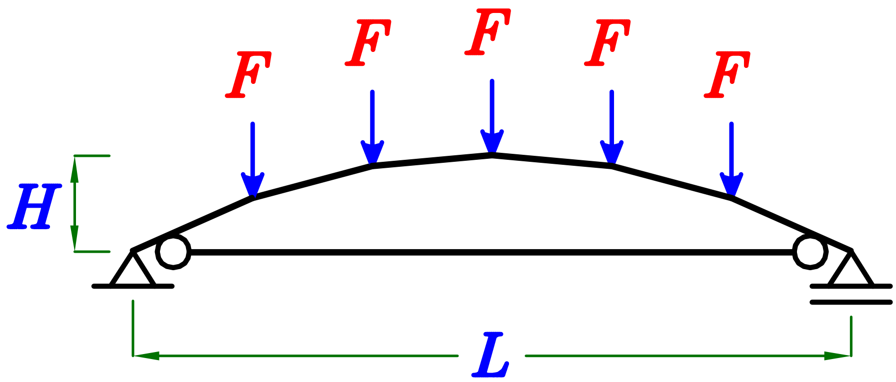
Beam B



$$w = o.w. + 2\,Y$$

$$R = w * S$$

Arch Girder

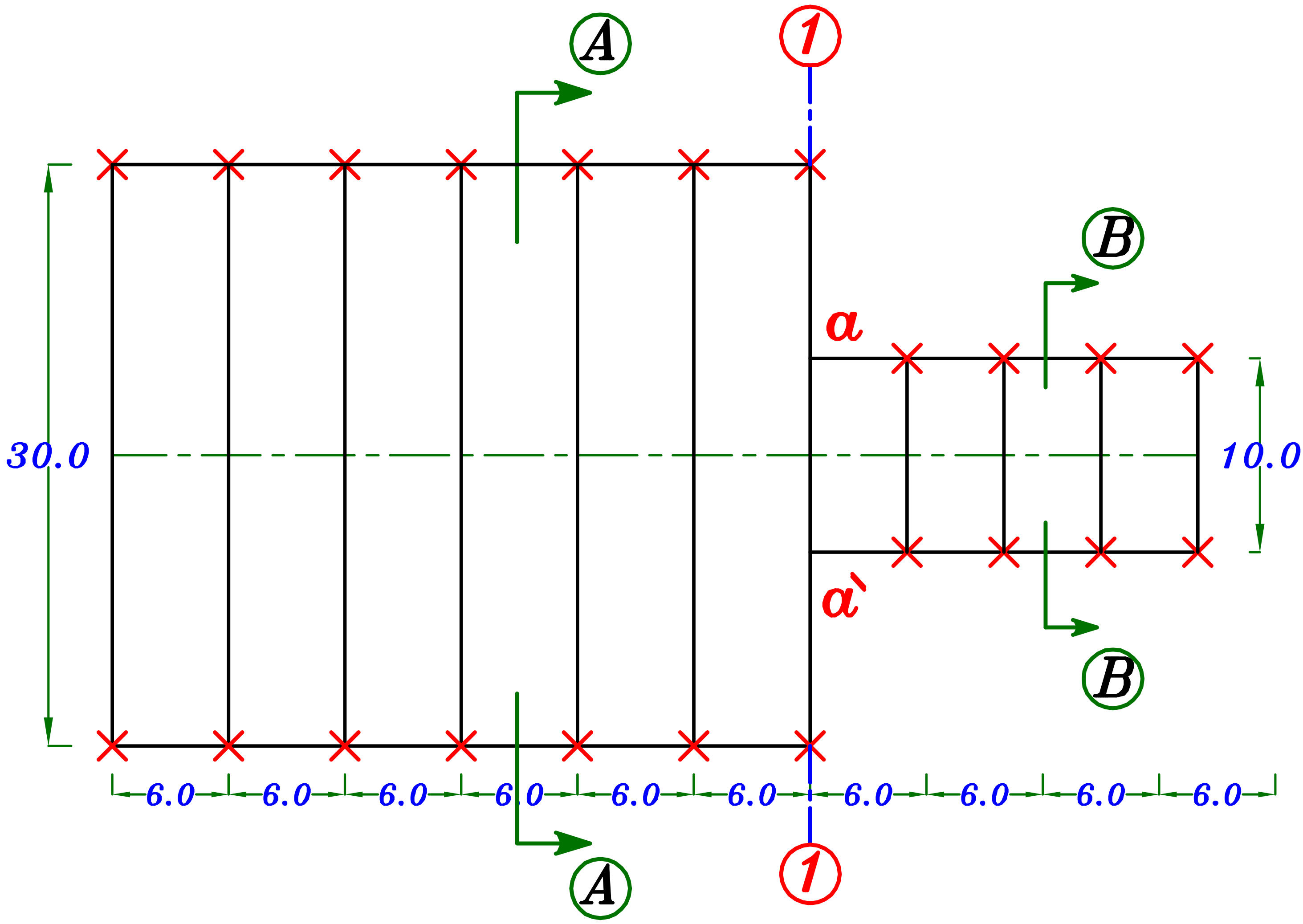


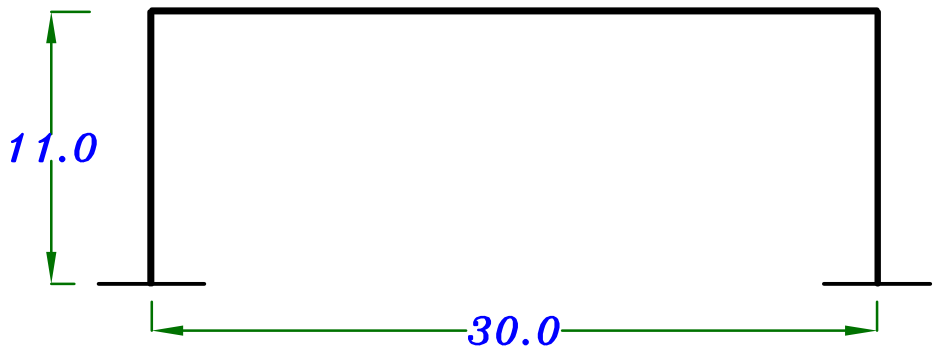
$$F = R + O.W.\,(\text{Arch Girder}) * \alpha$$

$$Tie = T\,(\text{Arch Girder}) + T\,(\text{Arch slab})$$

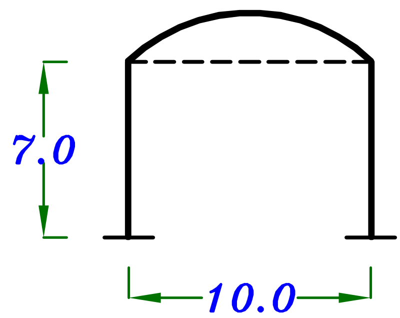
$$= 0.95\,\frac{M_o}{H} + X * S$$

Example.



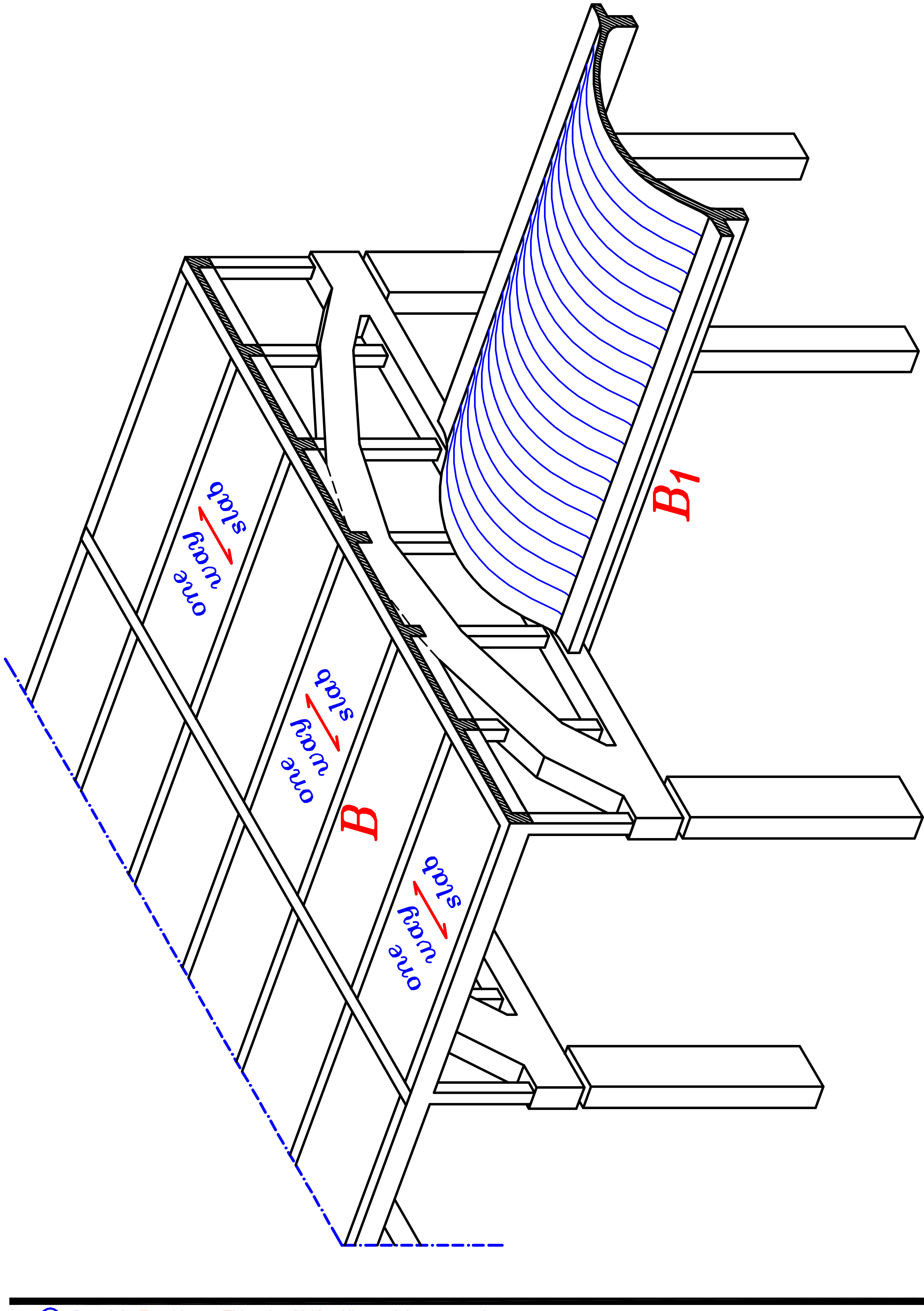


Sec. (A–A)

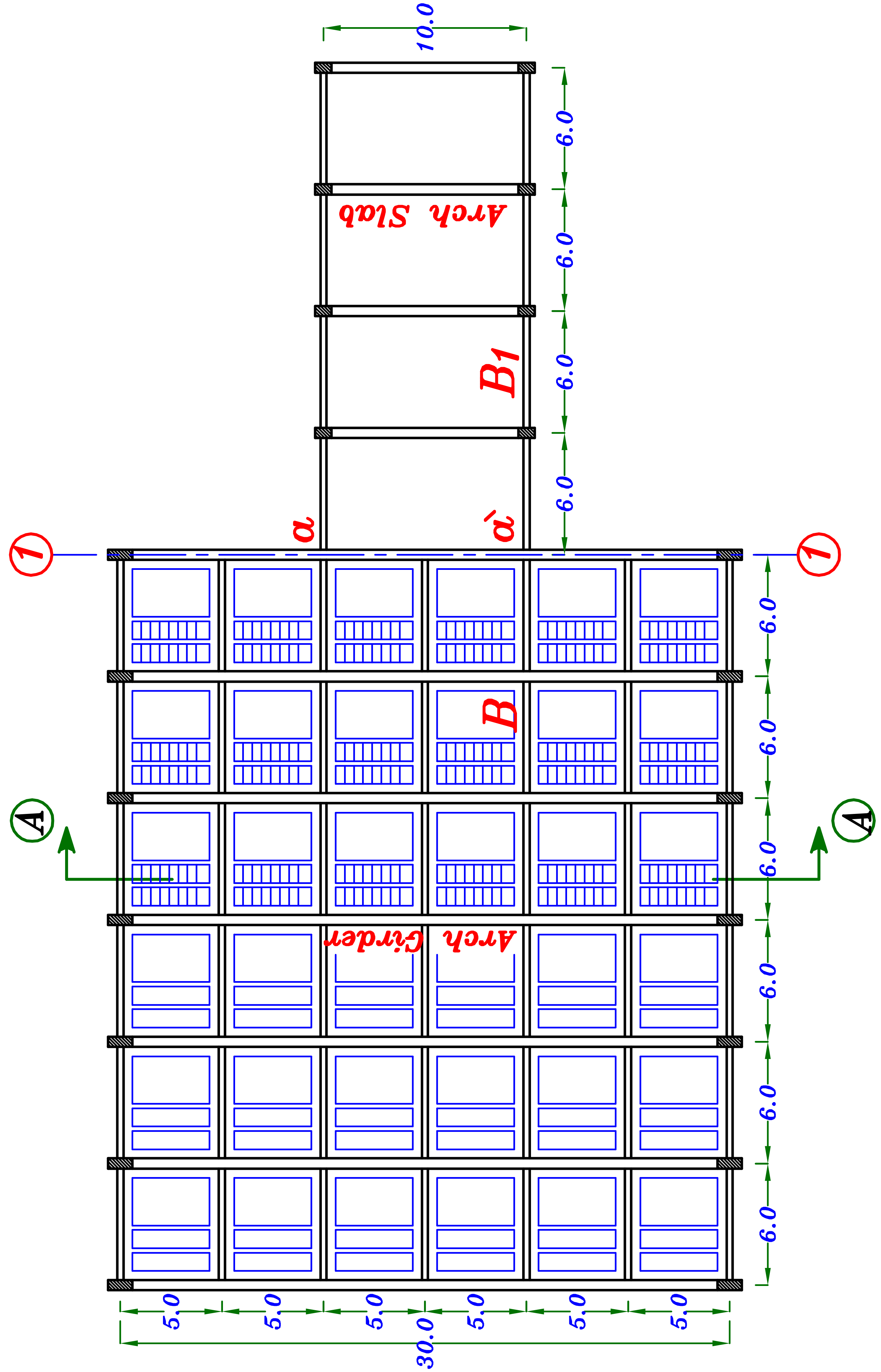


Sec. (B–B)

Design the system at axis 1



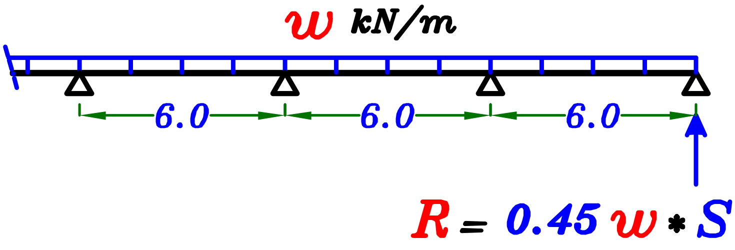
one way slab
one way slab
one way slab
B
B_1


10.0
6.0
6.0
6.0
6.0
Arch Slab
B1
α
α`
1
1
B
A
A
Arch Girder
6.0
6.0
6.0
6.0
6.0
6.0
5.0
5.0
5.0
5.0
5.0
5.0
30.0

Beam B

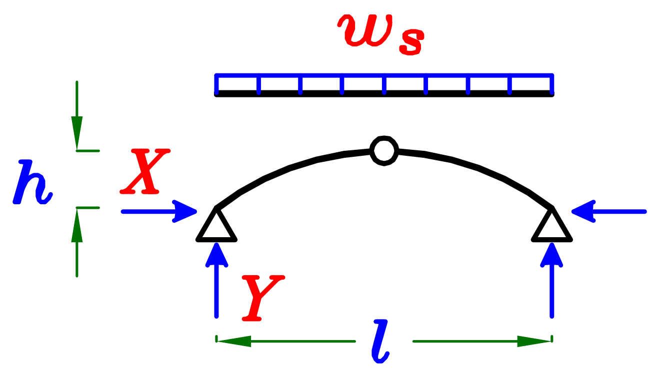
$$w = o.w. + \left(\frac{w_{rib}}{S}\right) * \alpha$$

$$R = 0.45\, w * S$$



Arch slab

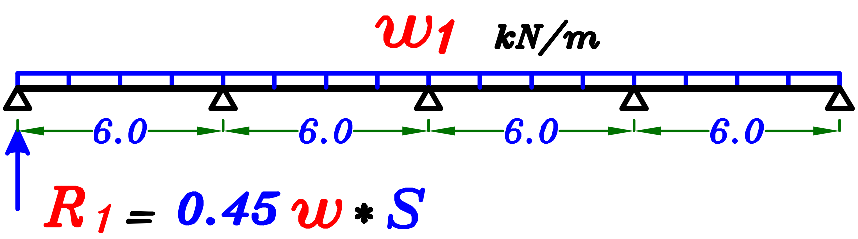
$$Y = \frac{w_s\, l}{2} \quad , \quad X = \frac{w_s\, l^2}{8\, h}$$



Beam B_1

$$w_1 = o.w. + Y$$

$$R_1 = 0.45\, w_1 * S$$



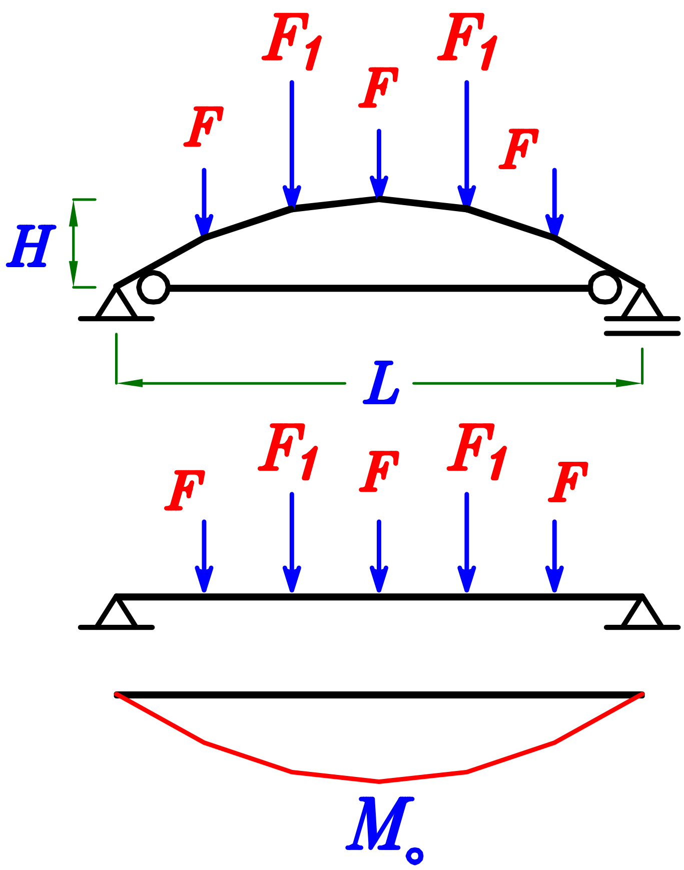
Arch Girder at axis 1–1

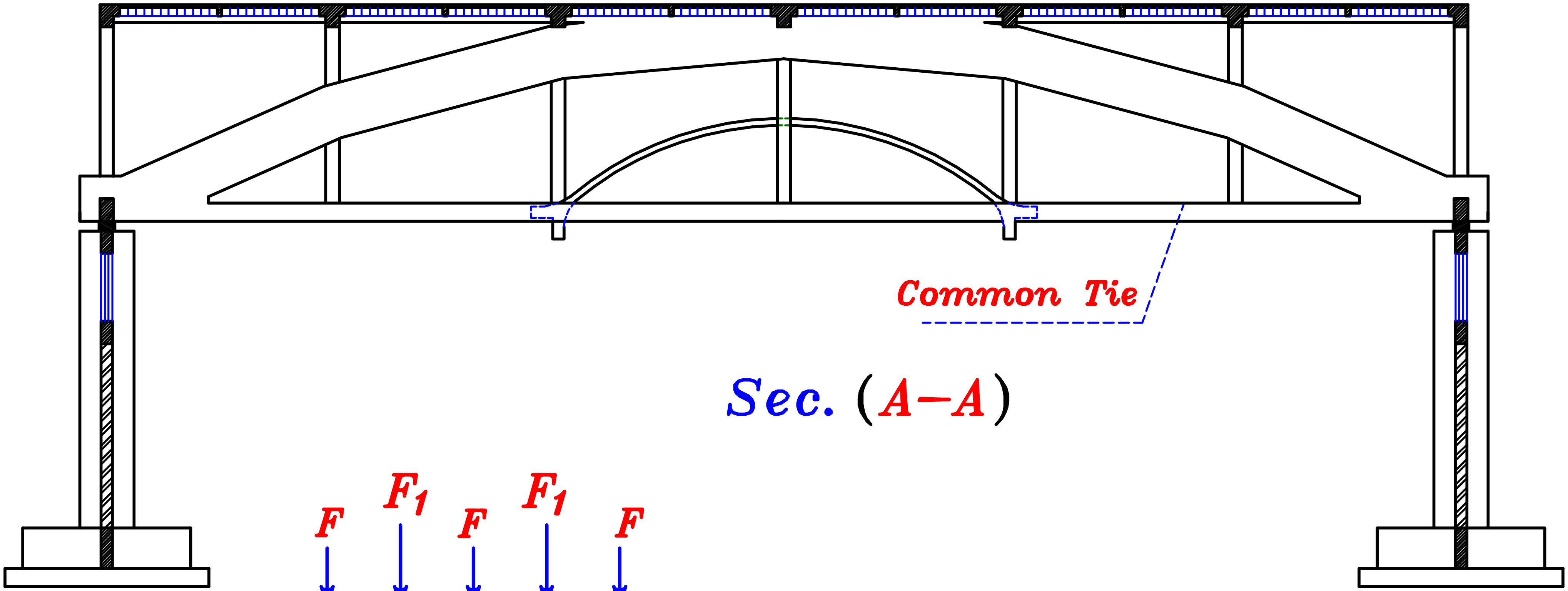
$$F = R + O.W. * \alpha$$

$$F_1 = R + R_1 + O.W. * \alpha$$

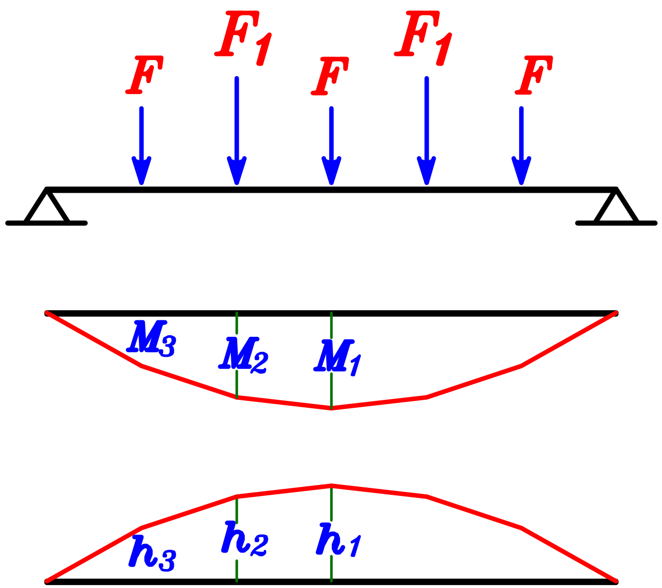
$$Tie = T(\text{Arch Girder}) + T(\text{Arch slab})$$

$$= 0.95\, \frac{M_o}{H} + X * S$$





Sec. (A–A)



Drawing Arch Girder at axis 1–1

عند رسم ال Arch Girder عند axis 1–1 الاحمال عليه غير متساويه لذا عند
تحديد ارتفاعاته لن ينفع أن نضع أحمال 1 kN عند كل Joint
لذا يجب أن نرسم ال moment الحقيقى أولا ثم نحدد الارتفاعات
على أساس أن نفس النسب بين الارتفاعات هى نفس النسب بين العزوم الحقيقيه